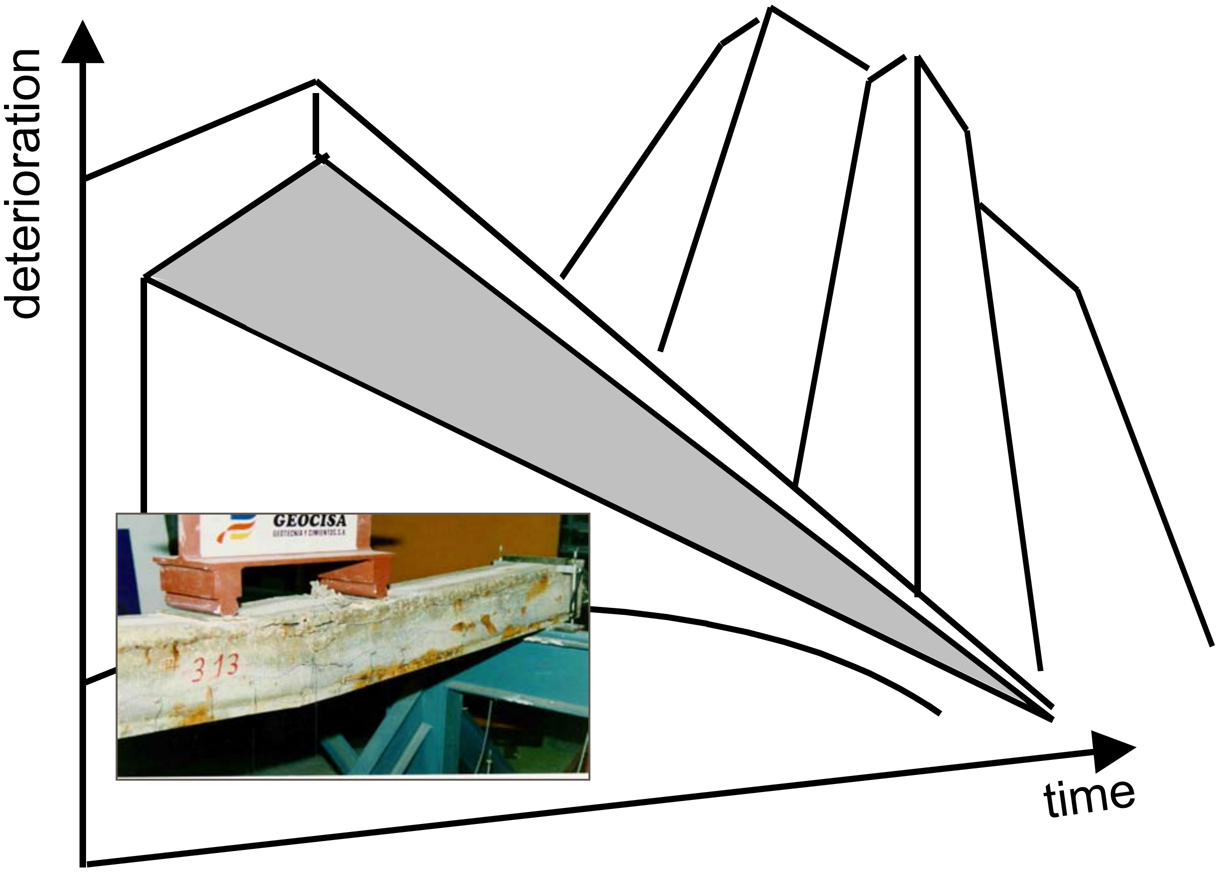
EC Innovation Programme

IN30902I

CONTECVET
A validated Users Manual for assessing the residual service life of concrete structures

Manual for assessing corrosion-affected concrete structures



GEOCISA
GEOTECNIA Y CIMIENTOS, S.A.

INSTITUTO EDUARDO TORROJA

EC Innovation Programme

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CONTECVET
A validated Users Manual for assessing the residual service life of concrete structures

Manual for assessing corrosion-affected concrete structures

BCA British Cement Association (UK)
GEOCISA Geotecnia y Cimientos S.A. (ES)
CBI Swedish Cement and Concrete Research Institute (SW)
IETcc Institute Eduardo Torroja of Construction Science (ES)
DGAV Dirección General de Arquitectura y Vivienda. Generalitat Valenciana (ES)
IBERDROLA (ES)
ENRESA (ES)
TRL Transport Research Laboratory (UK)
National Car Parks Ltd (UK)
Vattenfall Utveckling AB (SW)
Banverket (SW)
Swedish National Road Administration (SW)
Lund Institute of Technology (SW)
Skanska Teknik AB (SW)

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Information

Information regarding the manual

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1. INTRODUCTION

This Manual present an effective process of assessing concrete structures affected by rebar corrosion. Current version has been validated by means of its application to several *Case Studies*. The source information for the assessment concrete structures is based on an extensive testing plan developed for the Brite Euram project BE – 4062. Its final result was an assessing manual where many of the concepts here presented were established.

By means of the application to realistic *Case studies* during the CONTECVET project the manual have been calibrated and refined. Several corrosion indexes have been developed in order to represent the damage level: corrosion process and their effect in the structural load bearing capacity.

This manual is developed for structural specialists, who have expertise in the assessment field, with certain experience in reinforcement corrosion problems. It has to be stressed that a correct assessing of a structure suffering rebar corrosion, can only performed by a multidisciplinary team formed by corrosion specialists and structural engineers.

The content of the manual has been sorted in a main text body containing the operative procedures: The information and background are given in several annexes with graphical information.

Regarding the main text, two types of assessment methods are here proposed: a Simplified Method and a Detailed Method. Each assessment procedure the engineer is sufficient although the detailed method can be applied after the simplified one.

The number of annexes included in the manual are eight (numbered from A to F) and provide additional information on the background and basis of concepts required in the main text. The content for each annex is related above:

- **Annex A:** Fundamentals of corrosion, an extensive overview on the corrosion process and the explanation of how the corrosion is developed in concrete. Altso, a complete *state of the art* in mathematical and empirical models available for aggressive penetration calculations and prognosis are collected in this annex.

- **Annex B:** This annex presents the basic ideas about environmental classification and the source of aggressives in the environment. The classical EN206 exposure classification is collected here.

- **Annex C:** The basis for calculation the *Representative Corrosion Rate* to be used during assessment procedure are here presented.

- **Annex D:** The procedures for testing are related in this annex. Also the references to standards are provided.

- **Annex E:** Several notions and background on structural safety theory is here joined, in order to realise the partial safety factors methodology source.

- **Annex F:** Structural assessment.

Finally, it has to be remembered that this manual presents an approach to the assessment of concrete structures affected by rebar corrosion based in the experience and ideas of a group of specialists on both fields, structural assessment and corrosion process, the results expected using this manual have been checked and verified for their developers. Therefore, the procedure here proposed for the practical application must be taken into account as a recommendation and not a code. Thus, the procedure here described can be adapted or changed in order to fit within the requirement of assessing codes in each country under the responsibility of the assessing specialist.

2. LEVELS OF ASSESSMENT

As was explained in the introduction the assessment methodology is framed in two levels:

- *Simplified Method*
- and *Detailed method*.

Both of them are considered to be completely operational by themselves. The decision of use each type of assessment should be based on several criteria:

- Aim and importance of the assessment.
- Amount of elements to be assessed.
- Damage extension.
- Previous results of other inspections.
- Amount of information needed.
- Economical reasons.

Regarding *Simplified Method*, it is based on establishing a ranking of element performance, their actual state and a suggestion on the urgency of intervention. This methodology is specially suggested for owners with great amount of elements and structures to be quickly and efficiently assessed, using after it if it is required a detailed method for the assessment of each element It is also adequate for condominium owners or when it is necessary to make a preliminary assessment of a singular structure. It has to be stressed that the procedure has been developed mainly for common building structures and therefore their application to bridges and large structures should be applied with care. Thus, the *Simplified Method* should be seen as a procedure for establishing priorities in an extensive structural heritage, by means of a rational and quick process.

On the other hand *Detailed Method* has been developed for a rigorous assessment procedure by element taking into account the composite steel – concrete behaviour, as is common practice in structural designing procedures. Thus, the information and data required are, in fact, more numerous, all information needed for achieve the final safety margin of the element should be obtained from the inspection phase. This information must contain not only which regards structural performance, but also the information leading the corrosion process (corrosion typology, corrosion extension, reasons for reinforcement corrosion, etc.). This methodology can be applied in bridge elements (piers, beams, deck, abutments, etc.) and also in building elements and is based in the quantification of the reduction in load bearing section of the concrete and steel. The prediction of future evolution is based in the measurement of the steel corrosion rate.

3. SIMPLIFIED METHOD

The main tasks to be carried out during the simplified methodology are divided in three steps that are sequenced in time. 1) The first one is a complete inspection of the structure that allows to know the data needed for the input in the second task, 2) the assessment phase. Finally, 3) the prognosis about the structural performance is made through the data available from the assessment. Figure 3.1 shows the main steps and their result in the simplified methodology.

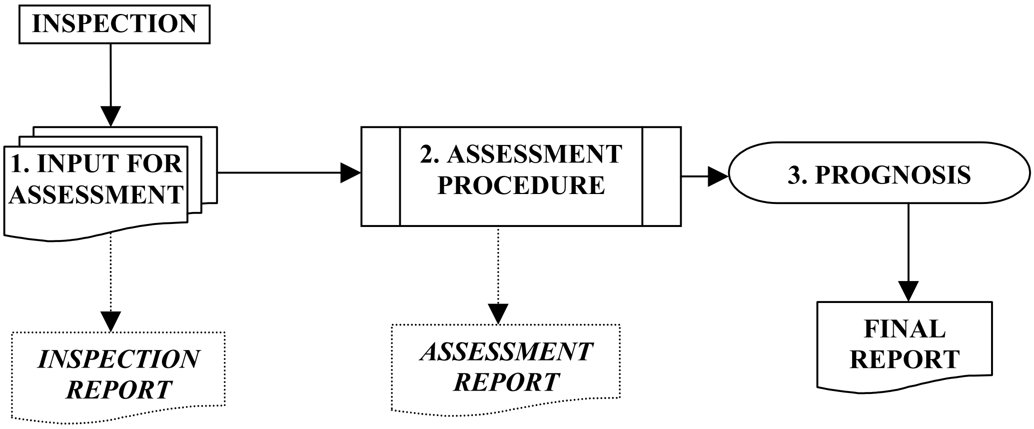


Figure 3.1 General overview of simplified methodology for assessment

The main objective of the <u>inspection</u> is to establish the cause of structural deterioration and the collection of enough data to be incorporated in simplified assessment procedure here proposed. As numerous countries have developed either an inspection procedure or a whole strategy for structural management, they may take advantage of the assessment procedure developed in present manual by fitting their procedures and strategies to the corrosion and structure indexes proposed.

The first step in using this manual is to find the damage mechanism that the structure presents. This manual is only leading with structures damaged by corrosion and therefore the procedure here related will be applied to the specific aspects that reinforcement corrosion includes in structural assessment. Regarding the <u>assessment</u> procedure itself, it is based in categorising three aspects:

- The exposure aggressivity.
- The level of material damage.
- The level of structural damage.

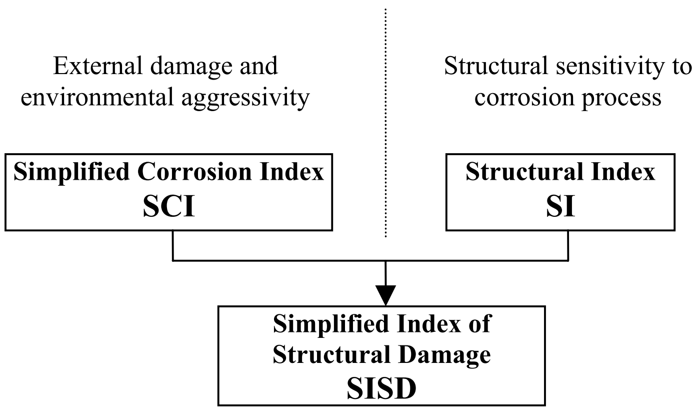


Figure 3.2 Calculation of SISD factor

The data related to these aspects are implemented into two main Indexes, the *simplified corrosion index SCI* which tries to represent the degradation level of the material, regarding the reinforcement corrosion and the *Structural Index SI* which represents the structural sensitivity to the corrosion process. Both factors are taken into account for the assessment of the structure in the *Simplified Index of Structural Damage* (SISD) which gives a comprehensive overview of the structural present state. The whole assessment procedure is summarised in figure 3.2.

The *SCI* is calculated:

- Through the establishment of four Corrosion Damage Levels, which are deduced from the registering of Damage Indicators (*DI*) ranked by means of weighing them from 1 (minimum damage) to 4 points (maximum damage) and,

- From making the same with the environmental aggressivity, *EA,* which is also ranked through the attribution of weights from 1 (most mild) to 4 (most aggressive).

The *SI* is a semi – empirical indicator, which takes into account: a) the structural sensitivity to corrosion and b) the effect of corrosion in the load bearing capacity of the structure. The *SI* index is calculated from:

- The *Rebar detailing* taking into account their sensitivity to reinforcement corrosion.

- The *Structural redundancy*.

- The *present load level* in the structure and their maximum load bearing capacity.

The joint consideration of *SCI* and *SI* aims into a Simplified Index of Structural Damage (*SISD*) which contains the final evaluation of present state of the structure ranked into four levels from negligible to very severe. From it, it is possible to calculate the urgency of intervention. Figure 3.3 shows a complete flowchart for the assessment procedure proposed in present manual, the main tasks of figure 3.1 are represented in the graphic.

3.1 ASSESSMENT PHASES

Two are the phases for the calculation of the *SISD*: Inspection and Assessment, being this last the one divided into Diagnosis and Prognosis. These phases are described in the following chapters.

3.1.1 Inspection phase

It aims into the collection of data necessary for calculating the *SISD*. The inspection consists on three main actions that can be developed simultaneously:

a) A Preliminary visual inspection
b) Desk work
c) In-situ testing

It has to be pointed out that the *In-situ* testing and the Visual (preliminary) Inspection can be merged in only one approach to the structure if enough information is supplied by the owner to make a previous complete survey.

3.1.2.1 Organisations with prestablished management strategy

Numerous countries and structures owners have their own procedure for inspection and management of structures which is based in their experience and available resources for the maintenance of structures. The inspection procedure here proposed can be completely skipped and merged into these management strategies. Thus, the inspection manual should only include those aspect no covered in the inspection phase that are needed for the determination of the *SISD* value. The *SISD* procedure is particularly well adapted due to its simplicity to procedures based in periodic inspections. The detailed method, further described, as well, but seems more appropriated for those cases where the information provided by the SISD can be considered as insufficient. The main aspects of structure classification (structural typology, damage level and expossure aggressivity) will be collected in the organisation database in order to actualise the graphical information about the state of their structures.

The sequence to be applied to obtain a representative information through the use of periodic inspections is represented in figure 3.3

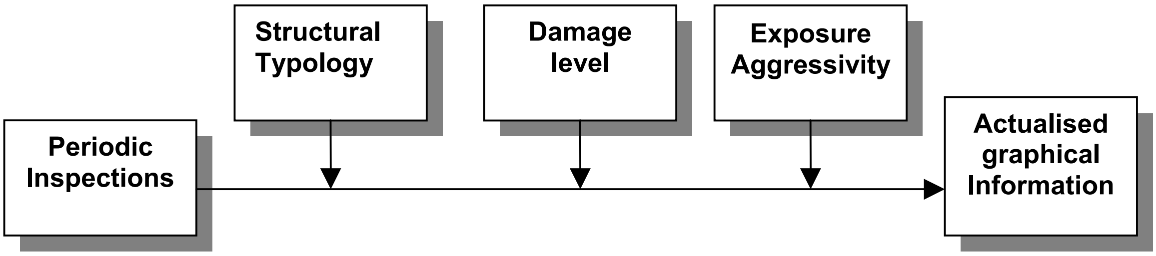
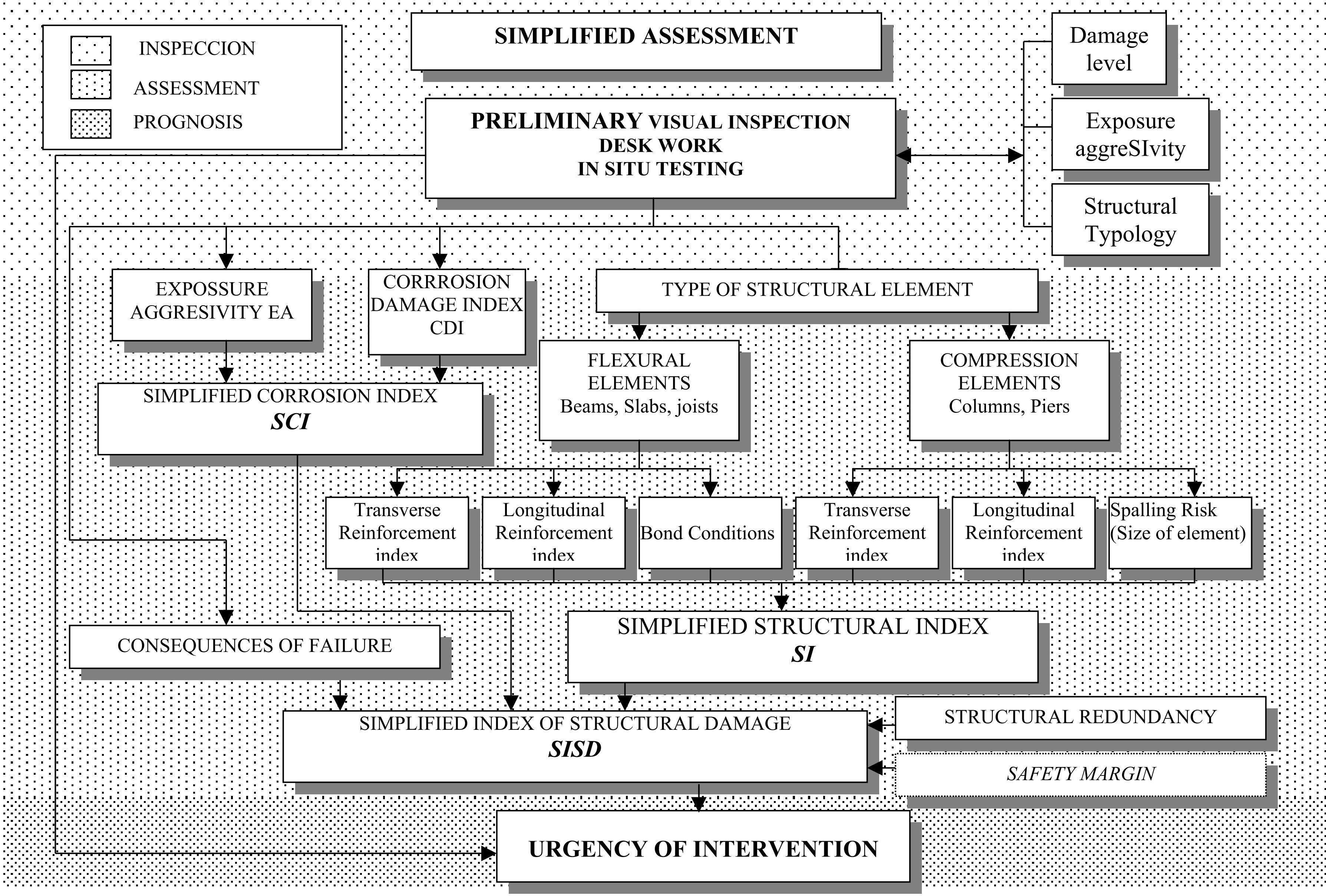


Figure 3.3 Relevant elements in the periodic inspection regarding reinforcement corrosion


INSPECCION
ASSESSMENT
PROGNOSIS
SIMPLIFIED ASSESSMENT
Damage level
Exposure aggreSIvity
Structural Typology
PRELIMINARY VISUAL INSPECTION
DESK WORK
IN SITU TESTING
EXPOSSURE AGGRESIVITY EA
CORRROSION DAMAGE INDEX CDI
TYPE OF STRUCTURAL ELEMENT
SIMPLIFIED CORROSION INDEX
SCI
FLEXURAL ELEMENTS
Beams, Slabs, joists
COMPRESSION ELEMENTS
Columns, Piers
Transverse Reinforcement index
Longitudinal Reinforcement index
Bond Conditions
Transverse Reinforcement index
Longitudinal Reinforcement index
Spalling Risk (Size of element)
CONSEQUENCES OF FAILURE
SIMPLIFIED STRUCTURAL INDEX
SI
SIMPLIFIED INDEX OF STRUCTURAL DAMAGE
SISD
STRUCTURAL REDUNDANCY
SAFETY MARGIN
URGENCY OF INTERVENTION

3.1.3 Preliminary Visual Inspection

It aims to survey the structure in order to detect which kind of damage mechanism is being developed in the structure, for the cases where reinforcement corrosion in the main cause of damage, then it is necessary to know:

1) Whether a corrosion process is or not developing, or

2) In the case the corrosion has already started: which is the level of structural damage.

For the sake of obtaining the *SISD* in the case of a simplified assessment or periodic inspection, three are the main aspects to be surveyed:

1) The type of structural elements
2) The environmental aggressivity
3) The level of damage

These three aspects will be further used to <u>group in lots</u>, if necessary.

Attending these three aspects the essential points to be investigated in the visual inspection are basically:

- <u>Structural typology.</u> Where adequate, the structural typology must be identified and classified in function of its elements. For instance, for bridges: abutments, main girder and secondary girders, piers and foundation elements.

  It is very helpful to use graphic information and therefore as a final result of the visual inspection, a simplified sketch of the structural typology should be performed in order to identify each element in the whole structure and identify sensible elements in the structural behaviour.

- <u>Identification of exposure aggressivity:</u> Several possibilities exist for quantifying the environmental aggressivity. For the sake of the *SISD* calculation it is proposed the use of the exposure classes given in EN-206 see (Annex H, table H.1.).

- <u>Damage level identification</u>: A preliminary classification of damages can be done regarding:

1) Damages due to structural behaviour: Such as transversal cracks in beams or inclined cracks due to shear effect.
2) Damages due to corrosion effect: Such as cracking, delamination, spalling, rust etc.
3) Damages due to expansive effects in concrete: Such as high width cracking, unmapped cracks.

Regarding reinforced corrosion problems, three types of damages are commonly found:

a) Rust spots from the corrosion products. The level of corrosion and its extension should be indicated.

b) Cracks due to reinforcement corrosion: cracks produced by reinforcement corrosion are usually parallel to reinforcement disposition both links, or main bars, therefore it can be easily identified and separated from structural cracks. In slabs with two ways reinforcement both types of cracks (structural and due to corrosion) *may be mixed, however in these cases is commonly found a dellamination of the surface. Rust spots are developed through the cracks and may be correlated with them.*

c) Spalling or loss of cover: when corrosion has been developed during some time, the pressure from rust products, make the cover to spall. Their location may be common on compression zones for bending elements or in columns.

Sketches of the type of figure 3.4 should be used to localise and identify each damage. In case of having a register of periodic inspection results the damaged surveyed may be actualised with every inspection performed.

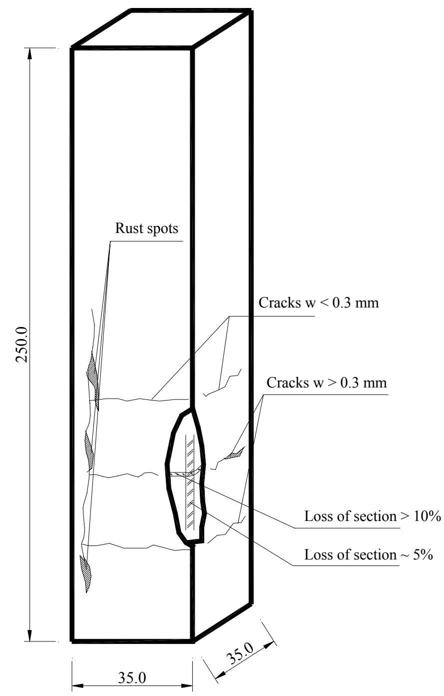


Figure 3.5 Sketch of damage in a pile

3.1.4 Desk work

The main tasks in the desk works are:

1. <u>Collection of background data on the structure.</u>

This information could be not available, however some information is needed in order to reduce the amount of data to be taken in the field, with its corresponding increasing level of costs.

Such an information needed should be:

a) The age of the structure-

b) Typology of the structure: not only regarding the main structure (multiple frames, slabs, hollow pot floors) but also their disposition in the structure in order to assess how the main gravity loads are transmitted to the foundations. In this step possible modifications with structural drawings should be noticed using the sketch of visual inspection.

c) Structural detailing of elements when *as built* drawings are available, this will reduce the *in situ* tests number and cost and will allow a better assessment of the structure ranking.

d) Amount of repairs performed during its service life and possible performance of them.

e) Periodicity of inspections and their results

f) Loading tests and results

2. <u>Identification of exposure aggressivity</u>

Exposure aggressivity should be established using the environmental characteristics of each element regarding corrosion and data available from visual inspection. Each element selected will be assigned to its corresponding environmental classification (see annex B).

3. Classification of types and extension of damages

A distinction between different damages will be performed. The scheme of types of damages and extension of them will be necessary on assessing the actual estate of the structure. In each lot the parameters constitutive of the indicators of damage type and corrosion indicators, selected to be surveyed for the calculation of the *CDI,* are:

0. Concrete microstructure features.
1. The depth of penetration of the aggressive (carbonation or chloride threshold) front, $\mathbf{X}_{CO2}$, X_{Cl}.
2. Cover thickness, ***c***
3. Cracking and spalling , ***Cr***
4. Presence of rust on the steel bar and, if any, bar diameter loss,ϕ
5. Measurement of the corrosion rate, $\mathbf{I}_{\mathbf{corr}}$, and
6. Resistivity, **ρ.**

4. Grouping in lots:

Finally, according to the different classification of structural typology, types of damages and environmental exposure, several lots of the whole structure will be grouped. In each lot a complete set of testing will be performed regarding material and electrochemical measurements.

The concept of lot assumes that all material properties and degradation rating derived from test in *in situ* works will be extrapolated at all elements formed by the lot. It is useful in large structures where it is necessary to realise that a discrete number of tests must characterise the whole structure.

3.1.5 In situ testing

In a simplified procedure few should be the number of tests to be needed *in situ*. It is said that the usually number of test per lot is three (in order to avoid any error of measurement). The tests that are considered to be necessary for the sake of present Manual are:

- Element geometry:

All elements studied must be surveyed in order to determine their geometrical dimensions, including cover thickness and rebar diameter if it is spalled. The use of dimensions will be needed on the assessment of present dead load.

- Material strength:

For those cases where the safety margin is going to be calculated, it is necessary a quantitative value of the material strength. Thus, three sources could be used for achieve calculation values:

- Test results obtained from cores or samples.
- Nominal values from *as built* drawings if they exist.
- Minimum nominal resistances provided by code of the same age of the structure.

- Reinforcement detailing:

If possible, reinforcement detailing for representative lots should be obtained. The use of classical pachometers will provide the location of reinforcement bars and stirrups (if they are present). If structural drawings are available the test will verify the location of reinforcement.

- Mechanism of deterioration and penetration of aggressives:

It is necessary to establish whether reinforcement corrosion is the only cause of deterioration or there are other process developing it could be done by means a microscopical observation of concrete. For carbonation process the phenolphthalein test will provide the aggressive front at present state. For chloride ingress into concrete, the best test is a whole profile using a core drilled in the element. If no core is available simple samples obtained from the element with a hammer will allow knowing the amount of chlorides that are present at different depths of the surface of the concrete.

- Corrosion measurements:

Two types of test are needed for achieving a *Representative Corrosion Rate* (Annex F): Concrete resistivity and corrosion current measurement. Both test values must be adequately combined in order to achieve a reliable value of *Representative Corrosion Rate* as is described in Annex E.

3.2. ASSESSMENT OF THE STRUCTURE

The assessment of the structure may be divided in two main aspects, the present estate, say *Diagnosis,* of the structure and its future evolution, say *Prognosis*. Basic models and equations are the same for both determinations although the *time effect* is only considered in the *prognosis* phase.

The purpose of the *diagnosis* phase consists in the appraisal of present performance of the structure in a simplified semi-empirical manner, based on the data collected during the inspection of the structure. In order to achieve this goal present manual, develops a methodology based on the establishments of a simplified index (*Simplified Index of Structural Damage)* ***SISD***.

The *prognosis* phase is established in present Manual as an Urgency of Intervention classification. If more information is requested such as final safety of the element or the effect of corrosion into the load bearing capacity of the element, the Detailed Method is more appropriated for the assessment.

3.2.1. Diagnosis of the structure

3.2.1.1. The Simplified Index of Structural Damage (SISD)

The *Simplified Index of Structural Damage* (SISD) is based on a simplified classification model that takes into account several aspects of the problem (environmental conditions, corrosion process characteristics and structural detailing characteristics). It also can establish whether a detailed assessment is necessary.

As mentioned previously (see figure 3.2) two main factors are used for the calculation of the *SISD* index, the sensitivity of the structural load carrying capacity to an active corrosion process and the process and damages presented in the structure. Both factors are calculated with the information available from the visual and *in situ* test. The *SCI* (Simplified corrosion index) tries to characterise the environmental aggressivity and the actual corrosion damage of the structure (*EA* and *CDI*). The *SI* value (*Structural Index*) provides and indicator on the structural sensitivity to corrosion.

3.2.1.2. Simplified Corrosion index (SCI)

The first is to identify the type of damage mechanisms developed in the structure. This is accounted in present manual by the collection of damage indicators (numbers 0 to 6). The microscipical observation of a concrete sample where the residual products of damage (ASR, frost attack or sulphates) can be identified. For each specific deterioration mechanism parts 2 and 3 of this manual are produced. The simplified corrosion index (*SCI)* represents whether the direct damage of rebars due to corrosion is progressing more or less rapidly. The corrosion process is ranked into four levels corresponding to the following criteria:

- N: No corrosion
- L: Low corrosion
- M: Moderate corrosion
- H: High corrosion

To achieve this classification the *SCI* index is based in two main corrosion factors: the environmental aggressivity *EA* and the actual damage state in the structure *CDI* (this is obtained from the *Corrosion indicators* numbered in 3.1.4 – 3)). If necessary, other *Corrosion indicators* can be selected in each particular case. For each one four weights have been selected as shown in table 3.1.

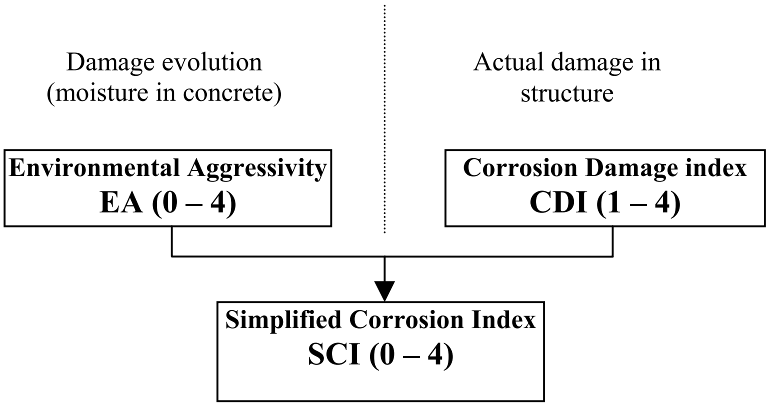


Figure 3.6. Elements of the simplified corrosion index

Table 3.1 Corrosion Indicators and Damage levels

Damage Indicators	Level I	Level II	Level III	Level IV
Carbonation depth	$X_{CO2} = 0$	$X_{CO2} < c$	$X_{CO2} = c$	$X_{CO2} > c$
Chloride level	$X_{Cl}^{-} = 0$	$X_{Cl}^{-} < c$	$X_{Cl}^{-} = c$	$X_{Cl}^{-} > c$
Cracking due to corrosion	No cracking	Cracks $w < 0.3$ mm	Cracks $w > 0.3$ mm	Spalling and Generalised cracking
Resistivity (Ω.m)	> 1000	500-1000	100-500	< 100
Bar section loss	< 1 %	1 - 5 %	5 - 10 %	> 10 %
Corrosion rate of main reinforcement (μA/cm^2)	< 0.1	0.1-0.5	0.5-1	>1

Where:

- X_{CO2} is the actual carbonation front in [m].
- X_{Cl} is the actual chloride threshold front in [m].
- c is the concrete cover in [m].
- w is the crack width in [mm]

For the exposure classes of EN206 table 3.2 presents the weight factors for exposure classification.

The C*orrosion Damage Indicator (CDI)* is obtained by averaging all levels obtained from inspection. In this case it is proposed to use the six indicators above (*n=6*).

$$CDI = \frac{\sum_{i=1}^{n} Corrosion\ Indicator\ level_i}{n} \quad (1)$$

Table 3.2 EA Weight factors for EN206

Class	*X0*	*X1*	*XC3*	*XC3*	*XC4*	*XD1*	*XD2*	*XD3*	*XS1*	*XS2*	*XS3*
Weight	0	1	1	2	3	2	3	4	2	3	4

The final calculation of the *SCI* is made, by averaging the weight of *Exposure Aggressivity,* table 3.2 with the actual corrosion damage indicators *CDI previously obtained (2).*

$$SCI = \frac{EA + CDI}{2} \quad (2)$$

3.2.1.3. Structural index

The structural consequences of rebars corrosion may be quite different depending on several characteristics of the element being assessed (main and transverse reinforcement detailing, gross concrete section, etc.), and depending also on whether the assessed element is mainly subjected to flexure or compression.

Accordingly, structural index definition is different for flexural and compression elements. In both cases, the structural index is a function of the characteristics of the reinforcement detailing, and the characteristics of the element affecting its capacity on flexure or compression respectively. Figure 3.7 shows the selected indexes to be taken into account for each structural element.

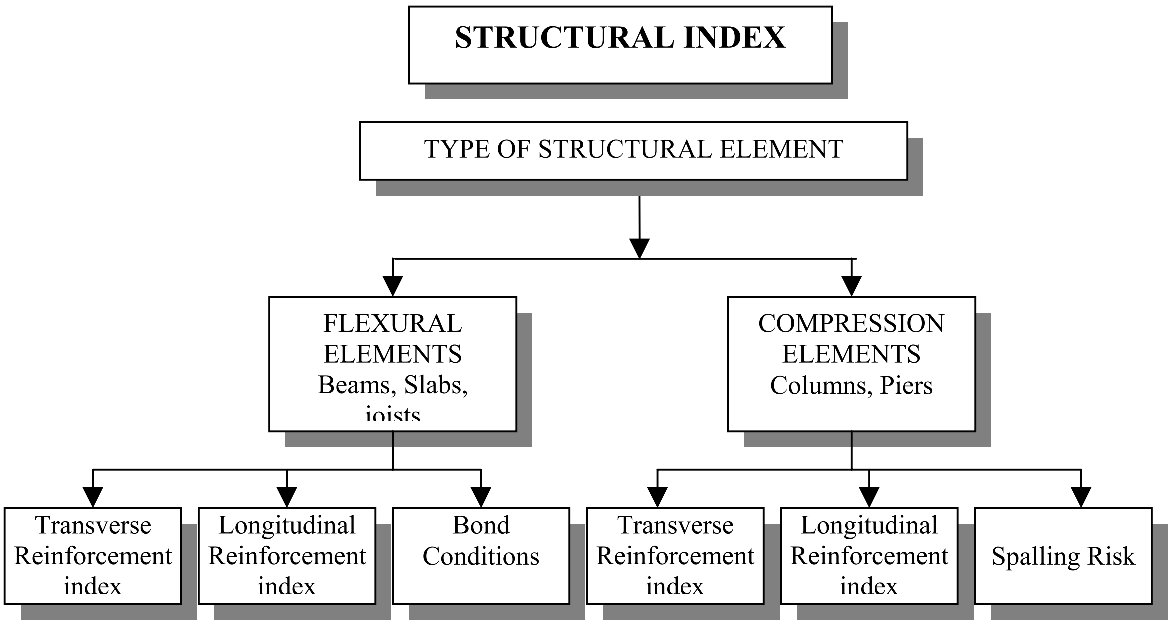


Figure 3.7. Structural index

a) Flexural elements (beams, slabs)

This index is suggested for classifying the transverse reinforcement detailing. The following parameters are taken into account in this classification:

- Diameter of the transverse reinforcement bars
- Spacing of the stirrups

These parameters are introduced in Table 3.4 in order to determine the general index for transverse reinforcement.

Table 3.4 Transverse reinforcement index (beams).

ϕ_t	**Stirrrups**			**No stirrups**
	$s_t \leq 0.5$ **d**	$S_t > 0.5$ **d (4 legs)**	$s_t > 0.5$ **d**	
>8 mm	1	1	2	1
≤ 8 mm	2	2	3	

Where:

s_T is the transversal spacing of the stirrups in [m].

d is the effective height of the element in [m].

ϕ_t is the transversal diameter of the stirrups in [mm].

Once that general index for transverse reinforcement has been obtained, the structural index for flexural elements is obtained entering table 3.5 with the categories of the main and transverse reinforcement of the element under consideration.

Two different categories of main reinforcement are considered depending on the diameter of the rebars:

- Large diameter: main reinforcement is formed basically by bars of $\varnothing \geq 20$ mm.
- Medium or small diameter: main reinforcement is formed basically by bars of $\varnothing <$ 20 mm.

Two subcategories are considered on each diameter category depending on the ratio of flexural tensile reinforcement, because beams of low reinforcement ratio would be more sensible to section reduction due to corrosion.

The ratio of flexural tensile reinforcement ($\varnothing_1$) would be considered to be low or high according to the following criteria:

- low for $\rho_1 < 1.0$ %
- high for $\rho_1 > 1.5$ %

For intermediate values of ρ_1, the assessment engineer would decide whether the structural index corresponding to high or low ratio columns should be applied.

If the ratio of flexural tensile reinforcement (ρ_1) is high, the ratio of compression reinforcement (ρ_2) should be taken into account because of the risk of spalling of the compression concrete cover. For ρ_2>0.5 the structural index should be the same corresponding to low values of ρ_1.

Table 3.5 Structural index (beams) (*)

TRANSVERSE REINFORCEMENT INDEX (**)	**MAIN REINFORCEMENT (mm)**			
	$\varnothing \geq 20$		$\varnothing < 20$	
	HIGH RATIO	LOW RATIO	HIGH RATIO	LOW RATIO
1	I	II	II	III
2	II	III	III	IV
3	III	IV	IV	IV

(*) Other variables to be considered:
Detailing of compressive reinforcement
(**) See Table 3.4

Structural index established in Table 3.5 corresponds to the normal situation in which some bars of the main reinforcement are anchored at intermediate points and could be more sensible to bond failure. If all the bars of the main reinforcement are anchored at support zones, in which it is reasonable to suppose that favourable conditions due to support external pressure exist, structural index should be obtained moving one column to the left.

If requested data for table 3.5 are not available, a preliminary simplification can be made. Thus, table 3.6 may provide the structural index for beams taking into account less information.

Table 3.6. Simplified structural index for beams

Transverse reinforcement index	**Flat beams (h < b)**		**Beams, slabs, joists**	
	Support section	**Mid span section**	**Support section**	**Mid span section**
No stirrups	---	---	I	II
High density stirrups	II	III	III	IV
Low density stirrups	III	IV	IV	IV

b) Compression elements (columns)

Again, an index is obtained classifying the transverse reinforcement detailing. The same parameters considered for flexural elements (diameter of the transverse reinforcement bars, and spacing of the stirrups) are taken into account now. Thus, spacing is analysed depending on the main bar diameter, in order to take into account the possibility of buckling of main bars due to corrosion failure of stirrups.

These parameters are introduced in Table 3.7 in order to determine the general index for transverse reinforcement.

Table 3.7 Transverse reinforcement index (columns).

	λ = stirrups spacing / $\varnothing$ main rebars	
$\varnothing_t$	**$\lambda \leq 10$**	**$10 < \lambda$**
>8	1	2
≤8	2	3

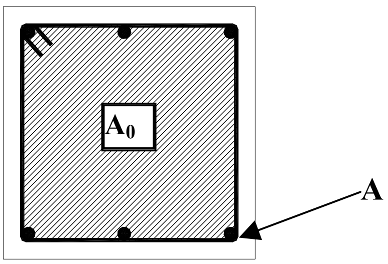
Once, general index for transverse reinforcement has been obtained, the structural index for compression elements is given by Table 3.8 in function of the category of the transverse reinforcement and some characteristics which reflects the sensitivity of the load bearing capacity of the element under consideration to concrete cover spalling. This is so because in some cases the load bearing capacity of a compressed element might be significantly affected if, due to corrosion deterioration, the concrete cover spalls and only the central part of the column is capable to resist the stresses.

This sensitivity of the element to concrete cover spalling is taken into account, through some reinforcing details (spacing of the main bars) and the characteristics of the concrete section of the element. This is taken into account through the following parameters:

- Ratio between the reduced element section if concrete cover spalls in all the perimeter, to the total original concrete section

- Spacing between bars of the main reinforcement. The closer are the bars the higher risk of concrete cover spalling.

Final reinforcement detailing index for compression elements (columns) is obtained entering Table 3.8 with the categories of the transverse reinforcement and of the spalling risk characteristics of the element under consideration.



$$\eta = \frac{A_0}{A}$$

Table 3.8 Structural index (columns).

TRANSVERSE REINFORCEMENT INDEX (*)	η = SPALLING RISK INDEX(*)			
	$\eta \geq$ 0.75		$\eta <$ 0.75	
	SPACING		SPACING	
	> 5 ∅	< 5 ∅	> 5 ∅	< 5 ∅
1	I	I	II	III
2	I	II	III	IV
3	III	IV	IV	IV

(*) The spalling risk index is defined as the ratio between the reduced element section if concrete cover spalls in the perimeter and the total original concrete section

Similar to previous section, where no information is available about the reinforcement. Thus, table 3.9 can provide structural index for columns taking into account the minimum data about the reinforcement.

Table 3.9 Structural index (columns) (simplified version).

Transversal reinforcement	**Minimum side dimension *a***			
	***a* > 400 mm**		***a* ≤ 400 mm**	
	High spacing vertical bars	Low spacing vertical bars	High spacing vertical bars	Low spacing vertical bars
Low spacing stirrups	I	II	III	IV
High spacing stirrups	II	III	IV	IV

3.2.1.4 Consequences of failure

The structural significance of an element is a function of the consequences of its failure. These are judged to be slight or significant as defined below:

- **Slight**: the consequences of structural failure are either not serious or are localised to the extent that a serious situation is not anticipated.

- **Significant**: if there is risk to life and limb or a considerable risk of serious damage to property.

The procedure, in which consequences of failure are taken into account when establishing the Initial Structural Severity Rating, is discussed in Section 3.1.2.7.

3.2.1.5 Structural redundancy

The existence or not of a certain degree of structural redundancy may change quite significantly the influence of a certain level of corrosion damage on the reduction of the load bearing capacity of the structural element under consideration.

For statically determinate structural elements the local failure of a section may result in the complete collapse of the element, while statically indeterminate structures may admit a considerable degree of efforts redistribution and, accordingly may be quite far away from a potential complete collapse when such local failure is reached.

The way, in which structural redundancy is taken into account when establishing the Initial Structural Severity Rating, is discussed in Section 3.1.2.7.

3.2.1.6 SISD Value

The classifications of the different material and structural indexes are combined in Table 3.10 to give an overall Simplified Index for Structural Damage (SISD) for each element as one of Negligible (n), Medium (M), Severe (S), and Very severe (V).

Table 3.10 Structural element severity rating (SISD).

SCI value	**SIMPLIFIED STRUCTURAL INDEX**							
	I		**II**		**III**		**IV**	
	Consequences of failure							
	Slight	Signif.	Slight	Signif.	Slight	Signif.	Slight	Signif.
0 – 1	*n*	*n*	*n*	*n*	*n*	*m*	*m*	*m*
1 – 2	*m*	*m*	*m*	*m*	*m*	*S*	*m*	*S*
2 – 3	*m*	*S*	*m*	*S*	*S*	*V*	*S*	*V*
3 – 4	*S*	*V*	*S*	*V*	*S*	*V*	*V*	*V*

SISD: *n: negligible, m: medium, S: severe, V: very severe.*

For each level of structural index two different columns are given depending on the consequences of failure already described in Section 3.2.1.5

The structural redundancy is taken into account, by means of an increment in the rating by one (e.g. from S to V). If the structure is a statically determinate one, which does not allow for any redistribution of efforts.

3.2.1.7. Safety margin index (optional index)

For those cases where a qualitative value of the structural safety and therefore more approximation is required, the safety margin index tries to take into account the sensitivity of the selected element to the design load of the structure. Thus, a possible reduction in the final classification of the element can be achieved. Two types of behaviours are selected, bending moment and shear (for beams) and axial load (for columns). The safety margin index is approximately equal to the safety factor for loads both on axial and bending moment (3). Thus, equations (3), (5) and (10) provide the ultimate section effort of the element.

$$SMI = \frac{M_U}{M_K} \quad or \quad \frac{N_U}{N_K} \quad or \quad \frac{V_U}{V_K} \qquad (3)$$

Where M_U or N_U are the ultimate section efforts and M_K and N_K are the characteristic efforts acting on the element. The influence of the safety margin in the main structural safety is fully dependent of the type of structure and loads, thus for each type of structure a subdivision between dead and imposed load should be done, however and as a first approximation table 3.11 provide a classification into three ranks that can be wide enough for covering most of the cases.

***Table 3.11. SMI** Value.*

Safety margin index	*LOW*	*MEDIUM*	*HIGH*
SMI	*1,4 < SMI < 2,0*	*2,0 < SMI < 3,0*	*SMI > 3,0*

*The way to take into account the load factor is by reducing the structural index in one or two levels as is explained in 3.3. **If the value of SMI is bellow 1,4 a whole structural assessment should be made on the structure due to its low safety margin**.*

The safety margin index will be calculated according to the element

1. *For beams elements the safety margin may be calculated as the minimum between the shear and the bending moment margin (4)*

$$SMI = min\left(\frac{M_U}{M_K}, \frac{V_U}{V_K}\right) \tag{4}$$

   The values of M_U and V_U must be calculated according to EN1992:1 (Eurocode 2) and using design values for the material strength.

2. *Safety margin index for compression elements,*

$$SMI = \frac{N_U}{N_K} \tag{5}$$

   The value of N_U must be calculated taking into account the effect of a possible existence of an external bending moment applied to the column, by means an interaction diagram.

The Safety margin index is taken into account, by decreasing the rating in one or two levels according to table 3.12

***Table 3.12**. Final SISD tanking into account safety margin index*

SISD	**Safety margin index LOW**	**Safety margin index MEDIUM**	**Safety margin index HIGH**
N	*N*	*n*	*n*
M	*M*	*n*	*n*
S	*S*	*m*	*n*
V	*V*	*S*	*m*

3.2.2. Urgency of intervention

For simplified assessment, the prognosis assessment is made establishing a ranking of urgency of intervention. If further information is required the detailed assessment will provide the time – decreasing load-bearing capacity.

Once the *Simplified Index of Structural Damage* (*SISD*) has been obtained (see Table 3.11), this classification, would allow to define the *Urgency of Intervention* entering Table 3.13.

Table 3.13 *Urgency of Intervention (years).*

SISD *value*	**Urgency of intervention**	**Action needed**
Negligible	> 10	Periodic inspections
Medium	5 – 10	Reassess structure during this time
Severe	2 – 5	Structural assessment within this time
Very Severe	0 – 2	Repair or detail structural assessment within this time

The type of intervention will differ depending on the case under study:

- For structures whose Urgency of Intervention, determined by means of Table 3.14, is higher than 5 years, recommendation is given to reassess the structure after that time, preferably after having monitored the actual corrosion rates in the structure.

- For structures which Urgency of Intervention is situated between 2 and 5 years, is recommended that a specialist consultant, in no more than that time carry out a detailed assessment.

- For structures whose Urgency of Intervention is lower than 2 years, most probably a repair action should be needed. The development of such repair project should previously require in most cases an immediate detailed assessment.

3.2.3. Assessment report

With all data collected through the detailed inspection and tests carried out, a report would be prepared containing the following information:

- Description of the structure: Structural typology, live and dead load supposed, element dimensions, type of foundations, etc.

- Definition of homogeneous groups of elements: taking into account the exposure and damage level.

- Description of characteristic damages observed for each group of elements: crack pattern, delamination, spalling,

- Diagnosis, and present state of the structure. Establishing whether those damages are produced by corrosion or not, and defining the characteristics of the corrosion process if it exists: origin of the corrosion (carbonation, chlorides, both), representative value of I_{corr}, E_{corr}, resistivity, carbonation depth, concrete cover humidity, present loss of section of rebars, etc.

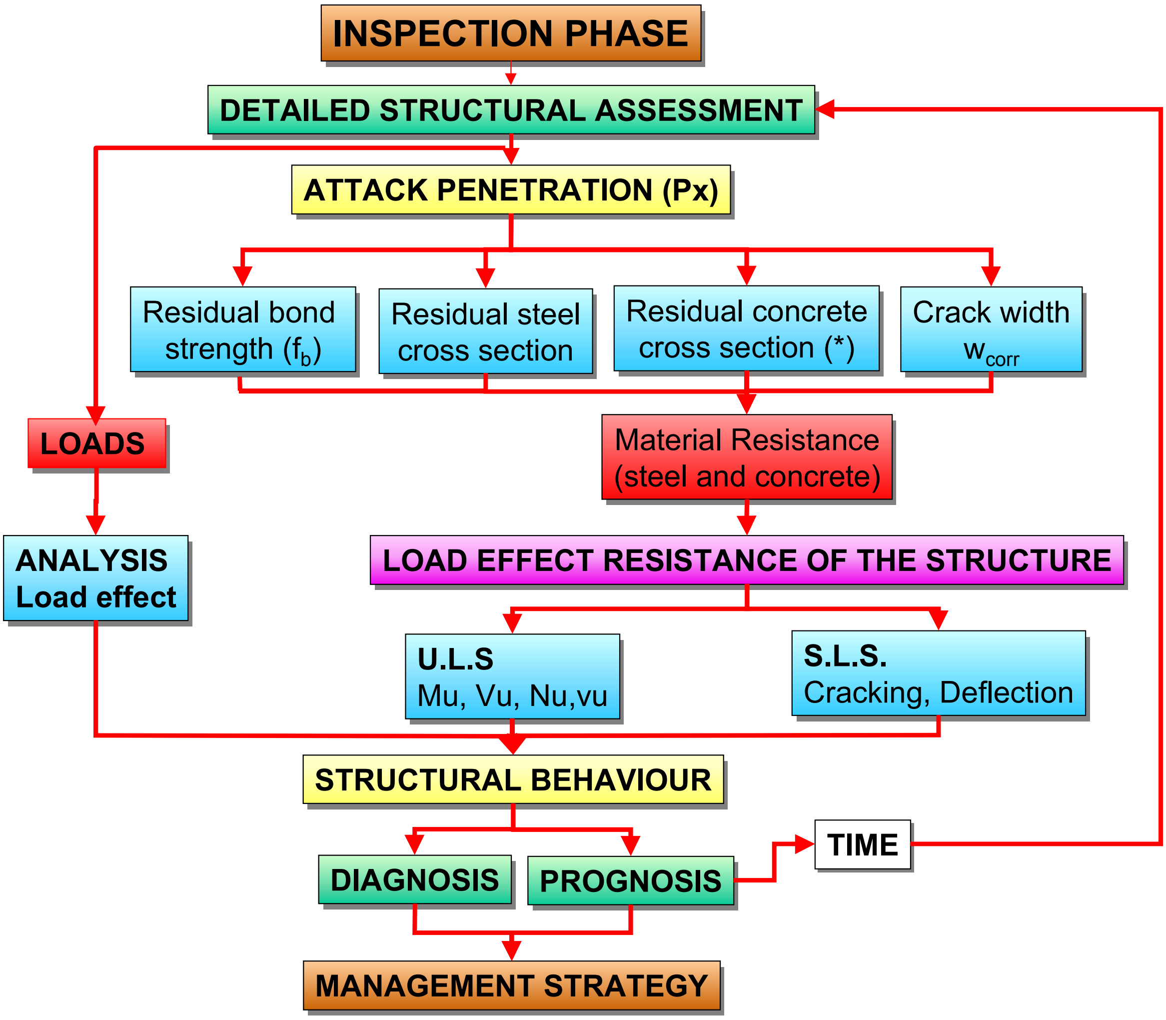
- Data for performing the structural evaluation described in Section 3.2: definition of concrete elements and reinforcement, mechanical characteristics of both materials.

- Calculation of *SISD* from the *SCI* index and *SI* indexes.

4. DETAILED METHOD

The detailed method considers material characteristics to be implemented into the structural behaviour in order to make a recalculation using the classical methods but considering the reduced resistance and sections.

The general process of the detailed assessment is exposed in the following diagram:



Five main steps can be distinguished:

1. Inspection phase, to collect all relevant data regarding the structure and its environment.
2. Determination of corrosion effects on concrete and steel, and specifically, on how bond properties, the steel cross section, the geometry of concrete section and cracking are modified by corrosion.
3. Load evaluation and analysis, taking into account the modified sections of concrete and steel.
4. Determination of the load effect resistance of the structure considering the new material properties.
5. Verification of the structural behaviour in both the present state (diagnosis) and in the future (prognosis) by means of the ULS and SLS Theories.

4.1. INSPECTION PHASE

When a detailed assessment is decided to undertaken, the information needed makes necessary a wide number of previous activities to collect all relevant background aspects for this kind of evaluation.

The needed information can be classified in different manners, depending upon the environment, material or structural characteristics. That presented in this Manual is shown in Table 4.1 and consists in:

- Characterisation of the environment (exposure classification)
- Concrete material characteristics needed to apply the deterioration models selected in present Manual.
- Identification of the causes of damage.
- The damage level and mapping found (mainly referred to cracking and deformations)
- Age of the structure and identification of the initiation period, t_i
- The permanent and live loads acting on the element. Although it should be theoretically possible, the consideration of all the variability and uncertainty of all the factors involved on the performance of a structure is so complex and expensive that it could be justified only in a very few cases. By the other hand, detailed assessment needs the evaluation of the dead and live loads that act on the structure to assess the residual service life of the structure. The dead loads should be accurately determined. Measurements of the geometric dimensions of the structural and non-structural elements of the construction should be carried out in order to estimate their self-weigh. The live loads should be assessed when it should be possible/necessary (change of use, etc.) Otherwise, they can be assumed as in the design phase of the structure.
- Structure characterisation: two levels must be considered for the correct understanding of the structure: the structure as a whole and its different components, including geometry, reinforcement detailing and material properties.

Three are the main steps to gather the information referred above: preliminary visual inspection, desktop studies and in-situ works. Obviously, if a preliminary assessment has been carried out before, some of the needed information should be avaliable. Table 4.1 shows the summary of procedures to be carried out in the assessment.

Table 4.1 *Procedures for assessment*

	Purposes	*Information needed*
Preliminary visual inspection	Identification of deterioration mechanism.	- Chlorides – Carbonation - Stress Corrosion Cracking
	Mapping of damages.	- Location - Aggressive front - Crack map, spalling, delamination - Section loss.
	Grouping in homogenous lots	- Type of structural element - Environmental aggresssivity - Level of damage.
	Selection sites for testing.	- Lots groups - Deterioration mechanism
Desk work	Collection of background data	- Calculations, structural models - History of events. - Age of the structure
	Exposure classification	- Climatic data. - Environmental actions: rainfall, chloride content, moisture.
	Grouping in lots	- Type of structural element - Environmental aggresssivity - Level of damage.
In situ testing	Testing for assessing	- Carbonation and chloride content - Concrete microstructure - Mechanical strength - Steel yield stress - Corrosion rate - Resistivity - Susceptibility to SCC
	Measurements	- Geometry and dimension of element - Loads on structure. - Rebar detailing - Cover thickness - Section loss

4.1.0. Preliminary visual inspection

The general approach adopted for detailed assessment of a reinforced concrete structure affected by corrosion of reinforcing bars, is reflected in table 4.1. The very first step on such an approach should be to identify whether a corrosion process is the only deterioration mechanism proceeding or not and if it is taking place or might occur in the future

Accordingly, visual inspection must be carried out for all different components of the structure and it would be focused on the detection of deterioration signs as colour and extension of rust staining, location and size of cracks or concrete spalling which could be produced by a corrosion process. If the detailed assessment is carried out as results of a preliminary assessment, the preliminary visual inspection could not be necessary as the information to be gathered should had been collected before.

Even if no visible signs that may be produced by corrosion are detected, the inspector should consider if the structure is in an aggressive environment, which can lead to corrosion in the future (presence of humidity together with carbonation processes or presence of chlorides in the concrete cover). The main aims of this inspection are:

- The identification of the main deterioration mechanism and whether other processes can be simultaneously proceeding.
- The mapping of the damages
- The preliminary selection of sites for posterior testing

4.1.1. Preliminary desk work

Simultaneously to the preliminary visual inspection, a certain work at the office is necessary.

4.1.1.1.Collection of Background data on the structure

If available, a great part of the relevant information concerning the structure is contained in the design and construction information. When approaching a structure, all information regarding repairs, modifications of the original design of the structure, maintenance operations, results of previous inspections, etc. should also be gathered. Thus, documents of special interest could be[4.1]:

- Calculations and structural models
- Design drawings
- As-built drawings
- Inspection reports
- Maintenance reports
- Repair reports
- Photographs
- Manufacturer's technical information, description of construction materials, etc.

There is generally a large volume of complementary information that can be useful of the structure assessment. This includes systems of constructions, textbooks and papers, codes for practice, etc.

4.1.1.2. Quantification of exposure aggressivity

Several possibilities exist for quantifying the environmental aggressivity in this Manual. The environmental classes considered as default are those included in EN-206, which are in Table 1 of Annex B.

It has to be stressed that other environmental classification are also possible (for instance national standard ones) better adapted to the particular case of the structure to be assessed can be also used providing it is coherent with the rest of concepts given in present manual.

In order to be able to identify the exposure class following EN 206, the aspects of environment that have to be well identified, during a preliminary Inspection are:

1. Whether chlorides are present or not. Three may be the sources of chlorides
   a) added in the mix (in the case of buildings and works before 70's or in zones where pure water is not available or clean aggregates are scarce)
   b) Added externally as deicing salts or being present in chemicals in contact with the concrete (industrial plantas, swimming pools, etc)
   c) marine environments

2. The distance of the concrete surface to the source of chlorides, which also means to the source of moisture

3. In the absence of chlorides Carbonation will be then the type of aggressive the regime of moisture in contact with the concrete is the main factor to be identified. At this respect the concrete can be
   a) In dry indoor conditions (interior of heated buildings)
   b) Sheltered from rain (interior of not heated buildings and outdoor exposure)
   c) Non – sheltered from rain and therefore subjected to cycles of dry-wet conditions)
   d) Permanently wet in contact with a source of moisture

4.1.1.3. <u>Grouping in lots</u>

Once a preliminary inspection has been carried out, the whole structure should be divided in different representative zones. Structural elements should be classified, forming lots (groups) of homogeneous elements according to three different criteria:

- Structural typology: flexural; compressed, massive and precast elements.
- Environmental loads according to the exposure classes.
- Damage level from the damage characterisation made during preliminary inspection.

Critical regions of the structure which are particular vulnerable to deterioration should be selected for more comprehensive investigation. In particular these areas may include:

- Areas subjected to high stress in service.
- Areas with potential weakness as a result of the construction procedures.
- Areas subjected to high environmental loads or particularly aggressive environments.

This classification is essential in order to establish lots of homogeneous elements, assuming that final decisions adopted can be different for different lots and will affect all the elements of the group.

As an example, considering supports of a bridge over the sea (figure 4.1), grouping in lots should be made considering the following exposure classes : XS1, XS2 and XS3. If there are also some supports with damages in the tidal zone, then four different lots should be grouped:

- The top part of the supports, exposed to XS1 and with no damages.
- The tidal zone of the supports, exposed to XS2 and with no damages
- The tidal zone of the supports, exposed to XS2 and with damages
- The bottom part of the supports, exposed to XS3 class.

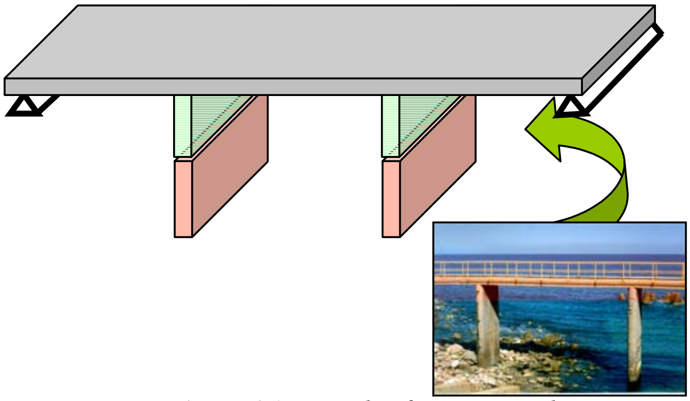


Figure 4.1 *Example of grouping in lots*

4.1.2. In situ testing

Once the preliminary desktop studies and the preliminary visual inspection have been carried out and all needs of information have been collected, a more detailed inspection of the structure should be planned.

It has to provide information about the structure as a whole and give the basis for making a complete characterisation of the previously established groups of elements in order to quantify (diagnosis) and delimitate the future performance (prediction). Accordingly, the general plan establishing the number and type of tests to be performed should specify the tests needed to obtain the parameters that would characterise each lot.

Depending on the type of assessment (simplified or detailed) to be performed, the engineer should establish in each case the extent and detail to be applied in each particular structure. The owners' requirements have to be well defined and in consequence, the aim of the assessment well clarified. In any case, the in-situ works aim into a careful diagnosis of present state of the structure and a characterisation by means of appropriate testing. A testing plan should be accomplished to define the number and type of tests that should be performed for each lot in order to characterise them.

This plan should include the objectives for testing and the influence of the expected results in the assessment procedure and in the prediction of their service life and residual load bearing capacity. As testing is an expensive process, a careful previous planning should be developed considering:

- Type of tests to be carried out
- The number of measures necessary to obtain reliable results
- The limitation of the testing procedures
- The location to obtain representative values.
- The needs of complementary devices to carry out the tests

The aim of testing is to gather information about those parameters that are relevant for the assessment of the surveyed structure and for the prediction of the deterioration mechanism's evolution. The tests needed to quantify the relevant parameters to be used for diagnosis and prognosis are listed next:

- Time of wetness or concrete water content
- Cover depth
- Carbonation and chloride depth

- Decrease in steel diameter
- Corrosion rate, resistivity and corrosion potential
- Steel yield point
- Reinforcement detailing
- Concrete mechanical strength

In the following Table 4.2 the parameters related with each test type are given

Table 4.2. *Relationship between tests and parameters*

Parameter	***Damage mechan.***	***Rate of penetrat CO_2 Cl^-***	***Propagation Period T_p***	***Steel corrosion P_X, P_{pit}***	***Crack width Spalling W, sp***	***Structual performance***
Cover depth		X				
Carbonation Chloride front	X	X	X			
Section loss				X	X	X
Corrosion rate	X		X	X	X	X
Water content			X			
Concrete strength						X
Yield strength						X
Loads						X
Geometry						X
Rebar detailing					X	X

4.1.2.1. Reinforcement detailing

There are three basic aspects concerning reinforcement detailing that should be known when an assessment is carried out :

- Concrete cover thickness
- Rebars' placement
- Rebars' cross section

The most common method of measurement of the cover thickness is the use of covermeters devices. Their operation is based on the different electromagnetic properties of the reinforcing steel, with regard to those of the surrounding concrete. This indirect method should be checked against direct observations made in exploratory removals.

Covermeters also allow to determine the location of both transverse and longitudinal rebars by passing the device through the structure and registering the changes in the magnetic field. This process is non expensive and non time consuming, so wide areas can be easily surveyed if access is available. A complete description of these methods is included in Annex D.

The number of exploratory removals depends on the structural typology and geometry of the surveyed element, but at least the more stressed zones of the element should be fully characterized (i.e. if a beam is assessed, support and middle span sections)

Measurement of the rebar diameter loss or attack penetration P_x can be made directly on the bar by previously eliminating the concrete cover and cleaning the oxides. Both longitudinal and transverse reinforcement diameters should be measured in sound and more damaged zones of the lot. As the diameter loss is not homogeneous, several measurements must be made, attending to measure the maximum or averaged loss.

If pitting is being produced, a calibrated wire should be used to measure pit depth P_{pit} in the exposed rebars.

4.1.2.2. Mechanical strength

In existing structures, core testing is the most common way to determine concrete strength. For compressive and splitting strength, core dimensions usually should be of 250 mm length and 100 mm diameter to obtain a length-diameter ratio of 2 after preparing the core for testing.

At least three specimens should be drilled from each lot in order to achieve a statistically representative value of mechanical strength. The places should be selected at random, but trying to represent the different zones of each lot. Attention has to be paid to the selection of core drilling when the concrete is cracked, in order to account for this damage.

Core testing can be combined with non destructive techniques as ultrasonic measurement or rebound and penetration methods to obtain a wider characterization of all parts of the lot (mapping of mechanical strength, etc.). Values obtained with these methods should be calibrated with drilled cores. A complete description of these NDT is included in Annex D.

4.1.2.3. Depth of aggressive front: carbonation and chloride advance.

To determine the depth of carbonation X_{CO2} a fresh concrete surface must be exposed. The advance of the carbonation front is determined by spraying the concrete surface with an acid-based indicator (phenolphtalein) that changes the colours according to the pH of concrete. At least four measures of non-coloured depth along the exposed surface must be carried out, including maximum and minimum values so a representative mean value can be obtained.

Carbonation depth can be measured on cores drilled for mechanical strength or in the exploratory removals. If it's not possible to drill a core or to extract a portion of concrete, a hammer drill can be used to obtain a fresh concrete surface exposed.

From the value of X_{CO2}, the V_{CO2} is obtained through the square root law:

$$X_{CO2} = V_{CO2} \sqrt{t}$$

where t is the age of the structure.

Regarding chloride advance, several methods can be applied to determine the total chloride content in hardened concrete. The investigations are made on dust samples taken from drill holes, drilled with a hammer drill, or directly scratched from the structure. The samples are taken from different depth-layers, measured from the surface, into the structure. When the cover is cracked or spalled, the fragments can be also taken for chemical analysis. The aim is to establish the chloride gradient or profile from the concrete surface to the interior and to identify the chloride threshold which produces depassivation.

Chloride profiles can be also obtained from cores. These cores are drilled from the structure and later scratched mm by mm.

Quantab-test and Rapid Chloride Test (RCT) are the most common methods to determine the total chloride content in field investigations. Other more accurate chloride analysis methods can be performed at laboratory. The chloride concentration can be expressed in several ways : as total Cl- percentage by dry mass of concrete or by weight of cement and as water soluble or free chloride content refered either to the concrete or to the cement mass.

Assuming a chloride threshold of 0.4 % by weight of cement or 0.1 % by concrete mass, the aim of this test is to measure the depth of penetration X_{Cl} of this amount, and consequently to calculate $V_{Cl.}$

$$X_{Cl} = V_{Cl}\sqrt{t}$$

4.1.2.4. Corrosion rate and complementary electrochemical parameters : resistivity and half-cell potential

Corrosion rate

The measurement of the corrosion current I_{corr}^{rep} gives the quantity of metal that goes into oxides by unit of reinforcement surface and time. The amount of oxides generated is directly linked to the cracking of concrete cover and the loss in steel/concrete bond, while the decrease in steel cross-area affects the load-bearing capacity of the structure. The rate of corrosion is therefore an indication of the rate of decrease of the structural load-carrying capacity

The most used technique to measure corrosion current is the so-called polarisation resistance, Rp, which is based on very small polarisations around the corrosion potential.

The measurement of the corrosion current is made by means of a reference electrode, which indicates the electrical potential, and an auxiliary electrode, which gives the current. In on-site measurements, a second auxiliary electrode (guard ring) modulated by two reference electrodes is necessary in order to confine the current into a limited reinforcement surface. Non modulating confinement techniques give too high values which overstimate the risk of corrosion. A complete description of these methods is included in Annex D.

The ranking of corrosion levels is given in table C.1 of Annex C.

Depending on several factors as the scope of the assessment, the type and location of the structure, etc., several strategies can be considered to obtain a representative value of I_{corr}^{rep} . The most common ways to achieve this goal are the following:

a) Use of nominal I_{corr}^{rep} values associated to exposure classes as indicated in table 4.3.
b) By means of on-site measurements. In that case two situations may happen : that continuous monitoring is made or that only a single visit can be performed.
    - In the case of continuous monitoring, the I_{corr}^{rep} is obtained from the averaging of the data recorded.
    - When only a single visit can be performed, an approach to obtain I_{corr}^{rep} can be made by averaging the value obtained on-site during the inspection (either from the diameter loss measurement or by the use of a suitable corrosion rate meter) $I_{corr,sing}$, with the value obtained from extrapolating the resistivity measured in a core to the I_{corr} value given by the I_{corr}-ρ diagram. The method is described in detail in Annex C.

As mentioned, when no measurement type of the I_{corr}^{rep} are feasible, the proposal is then to consider **Representative Corrosion Rate** values in function of the exposure classes. At this respect, those classes proposed in present version of EN 206 are here considered. Table C.3 of Annex C gives the proposed exposure classes and Representative corrosion rates[4.2].

Resistivity

The electrical resistivity of concrete gives information on the concrete water content and its quality and therefore is a useful complementary technique for locating areas of corrosion risk. A classification in resistivity levels is given in table C.2 of Annex C.

Concrete resistivity can be measured directly on the surface of the structure by Wenner technique and by the disc (one electrode) method. A complete description of these methods is included in Annex D.

Half cell potential

The main objective of potential measurements on a structure is to locate areas in which reinforcement is likely to be depassivated and hence, is able to corrode if appropriate oxygen and moisture conditions occur. The potential is measured by making electrical connection with the reinforcement and placing an electrode on the concrete surface. A complete description of these methods is included in Annex D.

According to ASTM C 876-91(1999) standard, a threshold potential value of –350 mV CSE can be established. Lower values of potential suggest corrosion with 95 % probability; if potentials are more positive than -200 mV CSE, there is a greater than 90 % probability that no reinforcement steel corrosion occurs, and for those potentials between -200 mV and - 350 mV corrosion activity is uncertain. Later practical experiences have shown that different potential values indicate corrosion for different conditions so absolute values can not be taken into account to indicate corrosion hazard, that is, the relationship between concrete condition and potential values is not well-defined enough, with the exception of those potentials at extreme ends. Therefore calibration has to be made in each structure.

4.1.2.5. <u>Yield strength and tensile strength</u>

The corrosion may induce changes in the mechanical properties of steel. When a detailed assessment is carried out, yield strength, tensile strength and total elongation at tensile strength should be known. With this purpose, if feasible at least one piece of reinforcement for each lot may be cut and tested. Nevertheless, due to testing difficulties and structural consequences, location and number of extract pieces should be conditioned to engineering criteria.

Loss in steel ductility and, although in many cases mainly in reinforced concrete, this is not a critical aspect, for very strong corrosion cases, it is recommended to find out whether the steel has become less ductile.. The strain-strength curve will indicate the mechanical parameters to be used in the recalculation and the likely loss in ductility.

4.2. STRUCTURAL ASSESSMENT

The minimum technical performance is the level of deterioration below which the structure or element should not be allowed to go. The level of minimum technical performance is likely to be set by national codes of practice for the ultimate limit state, where safety is the primary concern. For the serviceability limit state the level of minimum technical performance may be set by the structures' owners as the primary concerns will be aesthetics and function. Annex E gives information about structural safety theory.

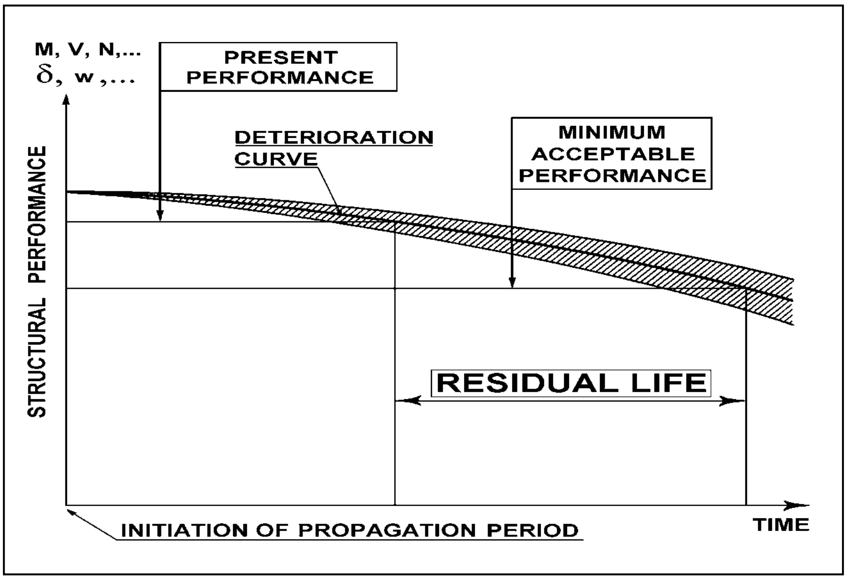


Figure 4.2. *Indicative deterioration of a structure with time*

The aim of this manual is to determine the performance of the surveyed structure at the time of assessment (Diagnosis phase) and to estimate how it will develop through a deterioration curve until it reaches the minimum acceptable performance (Prognosis phase). By this way, it is possible to estimate the residual service life of the structure (see figure 4.2).

The methodology to evaluate the current performance level of the structure and to carry out the prognosis is similar, and it is based on the following general guidelines:

- Eurocodes *'Basis for design'*, 1 and 2 have to be used for the evaluation. Chapter 2 *Basis for design* of Eurocode 2 has to be considered except when other requirements are indicated in the detailed method.

- Ultimate and Serviceability Limit States and Design Situations indicated in Eurocode *'Basis for design'* (section 3) will be taken into account. An additional SLS may be considered with regard to the external aspect of the concrete surface afected by some minor deterioration signs (rust spots, ...).

- Permanent and variable actions shall be evaluated:

- The permanent loads shall be accurately determined. Measurements of the geometric dimensions of the structural and non-structural elements of the construction shall be carried out in order to estimate the self-weight and dead load.

- The imposed loads shall be assessed when it is possible/necessary (change of use of the construction, ...). Otherwise, they can be assumed as in the design phase of the structure.

- Material properties have to be integrated into the structural performance. As it is shown in figure 4.3 consequences of corrosion on material properties can be classified in three main groups:

- Those that affect the reinforcement steel decreasing both the bar section and the ductility.
- Those concerning the integrity of concrete due to the tensional state induced by the expansion of rust that may lead to the cracking and spalling of concrete cover.
- Those affecting the composite action of both steel and concrete due to bond deterioration

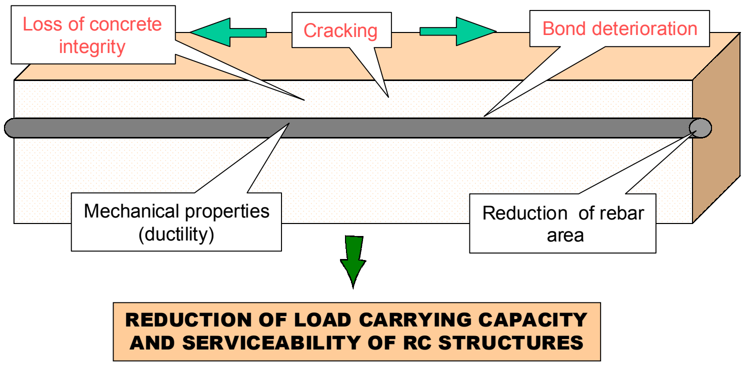


Figure 4.3. *Effects of steel corrosion on concrete structures*

The knowledge of the state and evolution of these three aspects is a decisive question to analyse the structural capability of the existing concrete structure and to estimate its future performance.

4.2.1 Method of analysis

The effect of the actions shall be obtained as it is indicated in chapter 5 of Eurocode 2 *Structural Analysis* but considering some aspects:

- Concrete section shall be modified to take into account the loss of reinforcement cross sectional area and concrete delamination and spalling.

- The ductility of the RC section is reduced, because corrosion reduces the elongation at maximum load in the reinforcing bars and corrosion in compression bar cracks the concrete at compression chord and reduce the effective depth. Thus, it is suggested to limit the ratio δ of the redistributed moment to the moment before redistribution when calculating the moment using linear elastic analysis, as it is commented in Annex F. The non-linear analysis shall be considered with caution due to the reduced ductility of the deteriorated reinforced concrete section.

- More rigorous alternatives will be used when they were available/necessary.

It is recommended that a linear elastic analysis be undertaken for the assessment of corrosion affected structures much as it would for a conventional structural assessment. In the case of bridges, this is likely to be a grillage analysis, whilst for buildings it is likely to be a plane frame analysis.

Although linear analysis in no way simulates the behaviour of reinforced concrete structures as they approach their failure loads, it does lead to safe designs and assessments. It achieves this by providing a set of stresses which are in equilibrium with the applied loading. If the member resistances are greater that these equilibrium stresses then the lower-bound theory of plasticity can be invoked. These states that provided a structure is in equilibrium with the applied loading and the yield stresses are not exceeded anywhere within the structure, then the structure will not fail at a lower load. As such, structures assessed to be satisfactory on the basis of a linear elastic analysis can be assumed to be safe. However, the converse is not necessarily true, as structures tend to have a higher reserve of strength than might otherwise be apparent from a linear elastic analysis. In such a case, a more sophisticated analysis may well be appropiate. Outline guidance on alternative methods of analysis is given in later sections.

4.2.2. Section properties

Section properties shall be considered to obtain the effect of the actions (analysis) and to verify the Limit States (ULS and SLS). The comments in this section deal with the section properties to be considered in the analysis, whereas the considerations for the verification at the ULS and SLS are commented in Annex F.

Conventional assessment codes such as BD 44/95[4.4] in the UK, allow the use of section properties based on:

i) concrete section (uncracked concrete no reinforcement);
ii) gross transformed section (uncracked concrete plus reinforcement); or
iii) net transformed section (cracked concrete plus reinforcement).

In design, the aim is to determine the amount of reinforcement required and so section properties based on the concrete section are an obvious choice in order to avoid an iterative design process. However, in assessment the reinforcement quantities and locations are known and so, the use of either of the transformed sections is now possible.

If members in an indeterminate structure are cracked then cracked section properties should be used. As such, they will have lower relative stiffness and attract less load. A consistent approach should be adopted which reflects the different behaviour of various parts of the structure. However, there are certain anomalies here. For instance, in older bridges, deck slabs were only lightly reinforced transversely and are therefore likely to crack relatively easily. It would thus seem reasonable to use cracked section properties throughout at the expense of reducing transverse load distribution. In such a case, the deck slab may well be satisfactory but the longitudinal members may appear to fail as they are unable to shed load transversely. If uncracked section properties were used then the situation may well reverse due to enhanced transverse load distribution. Jackson suggests that although intermediate section properties are not explicitly allowed, it is illogical to allow the two extremes (cracked and uncracked) without allowing intermediate section properties. It may well be useful to use intermediate values where different members in a structure alternate between passing and failing with cracked and uncracked section properties.

The section properties used should also take into consideration the assumptions made in calculating resistances. For instance, if the cover concrete is to be ignored in calculating the capacity of a column, then it should also be ignored when calculating the section properties in order to maintain consistency.

There are no definite guidelines for selecting section properties for corrosion affected structures. It is thus likely that a series of analyses will have to be carried out with different section properties, which are indicated in Annex F, to test assumptions and investigate the sensitivity of the structure.

4.2.3 Partial safety factors

The safety factors used in design are, in part, intended to guard against any deviations from characteristic material strengths or applied loads. At the assessment stage, much more is known about the structure, its constituent materials and the applied loads than at the design stage. This implies that those components of the safety factors which relate to possible deviations from the characteristic may be reduced as we have better estimates of what the material strengths and applied loads are for the structure. Annex E gives information about structural safety theory.

The reduced safety factors should only be used if the engineer is confident that his assessment is sufficiently rigorous that many of the uncertainties encountered at design stage are now known.

Greater reduction in loading actions may be obtained by quantifying the applied loads more accurately and assessing against those loads. This is likely to be more applicable to buildings than bridges as building loads can be readily assessed and measured where as bridge loads are specified in national standards as loads which the bridge must be able to carry.

4.2.4. Ultimate limit state

At the ultimate limit state, the aim is to satisfy inequality [4.1]. That is the structure must be able to carry the loading applied to it.

$$\mathbf{R \geq S} \qquad [4.1]$$

Where: R is the assessed resistance; and
S is the assessed loading.

National codes of practice specify both the levels of loading that should be applied to a structure and the means of calculating the resistance of members. For bridges, it is important that the minimum technical performance corresponds to the type of vehicles using the bridges. As such, there is no possibility for relaxing the level of minimum technical performance via the loading side of inequality [4.1] unless the weight of the vehicles using the bridge is to be limited. For buildings, loads are given in codes of practice but the building may undergo a change of use and so the required loading appropriate to the use of a particular building should be established with that building's owners. The factors of safety applied to the loads may be reduced in line with the guidance given in 4.2.3. This will appear to lower the minimum technical performance level, but in reality the level will remain approximately the same due to a reduction in the number of unknowns between the design and assessment stages. Thus whilst the applied loading effects are reduced and the member resistances increased, the overall reliability will remain approximately the same.

4.2.5. Serviceability limit state

At the serviceability limit state the engineer has greater flexibility to reduce the level of minimum technical performance provided that there are no safety risks associated with this reduction.

If the owner of the structure is willing to accept greater deflections or crack widths than those specified in current codes of practice then the minimum technical performance level will effectively be reduced. Such reductions are likely to be possible in structures where aesthetics are not so important. That is, cracking which may be acceptable in an industrial structure will not be acceptable in a prestigious office block.

The implications of accepting a reduced minimum technical performance level could include a shortening of the residual service life (as cracks can provide access for moisture and oxygen) and deflections that can induce second-order stresses. However, such implications have to be considered in the light of an overall management strategy. Accepting a lower level of minimum technical performance can mean postponing repairs to a more convenient time, but they may need to be more extensive.

There can be safety risks associated with lowering the minimum technical performance level. These may include spalling concrete falling onto people or excessive deflection leading to failure by an alternative mechanism. The risks associated with a lower minimum technical performance level must be investigated before such a level can be accepted at the serviceability Limit State.

No firm guidance on a general minimum technical performance is provided in this manual as each owner and structure may have different requirements. The best approach will be to investigate a range of minimum technical performance levels and select the most appropriate one based on safety, function and cost and agrees this with the owners, relevant authorities and insurers.

4.3. DIAGNOSIS PHASE

The aim of the diagnosis phase is to determine the present performance of the structure in order to locate where is the structure along the deterioration curve.

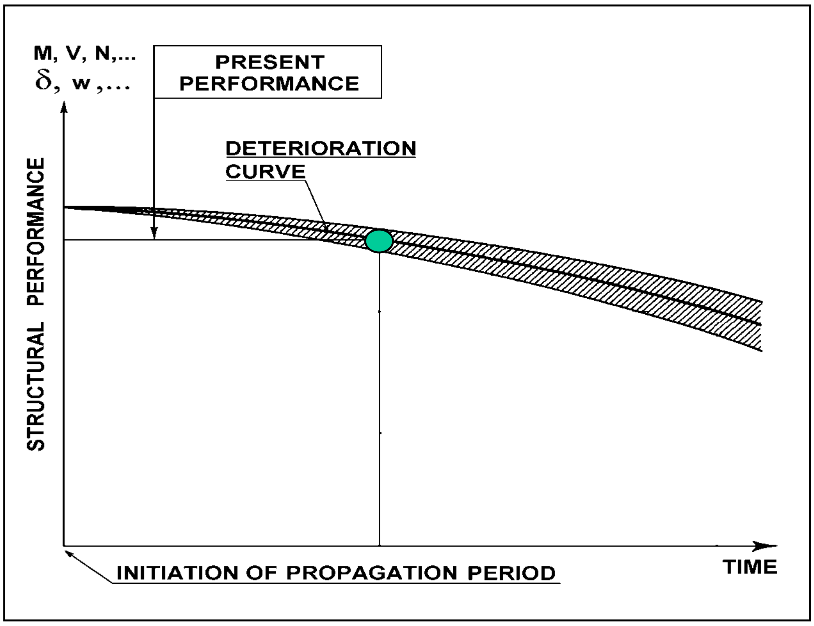


Figure 4.4. *Present performance of the assessed structure*

The steps to assess the present state of an affected structure are the following:

1. Identification of the damage mechanism.Characterization of the mechanism of attack and of the quality of the concrete are preliminary essential points of the diagnosis.

2. Measurement of the penetration of carbonation and chloride thresholds, and calculation of V_{CO2} and V_{Cl}. The measurement of the depth of the aggressive front, X_{CO2}, X_{Cl} is described in Annex D.

3. In the propagation period, estimation of time t since it started by means of the 'square root of time' equation described in 4.3.1.

4. Determination of corrosion attack, penetration P_X and the representative corrosion rate.

5. Determination of reduced cross section area of rebars, cracking of concrete cover and bond deterioration as it is shown in Annex F.

6. Application of ULS and SLS theory taking into account the special considerations for corroded structures described in Annex F.

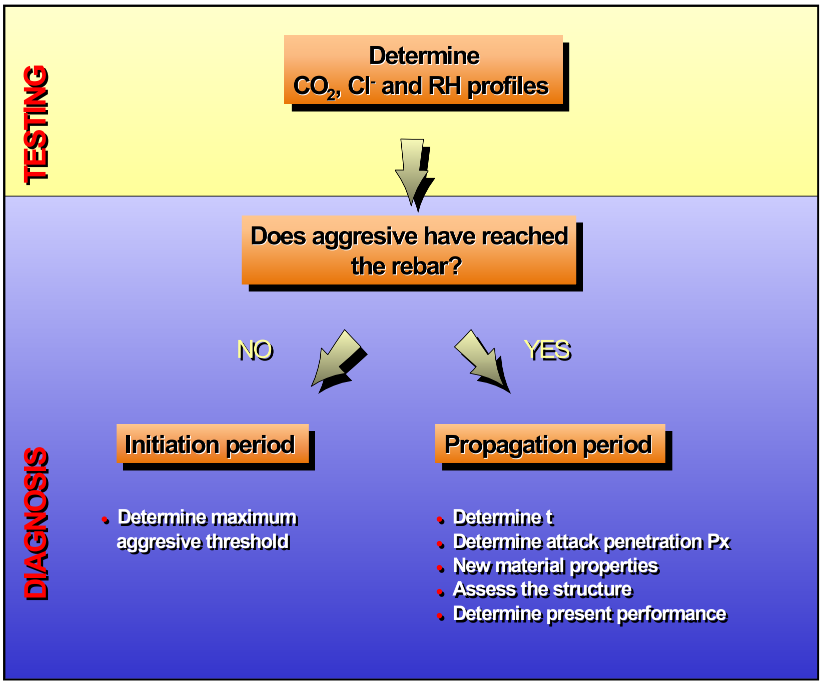


Figure 4.5. *Diagnosis process*

4.3.1. Rate of advance of aggressives and determination of the propagation period

The rate of advance of the aggressive front can be determined by using the 'square root of time' equation:

$X_{CO2} = K_{CO2} \sqrt{t}$ (Carbonation)
$X_{Cl} = K_{Cl^-} \sqrt{t}$ (Chlorides)

where X is the aggressive depth and t is time since the structure was exposed to the aggressive environment.

There're also other more **refined** [4.5,4.6] methods based on the more rigourous calculation of the diffusion processes involved on both carbonation or chloride penetration. They are reflected in annex A.

Determination of the propagation period t_p

Once propagation period has started, the calculation of the previous corrosion attack can be made by making a back extrapolation from the measured aggressive front depth to calculate the elapsed time since it reached the rebar, t_p . This time can be obtained by using the 'square root of time' model. Fig 4.6 shows this back extrapolation.

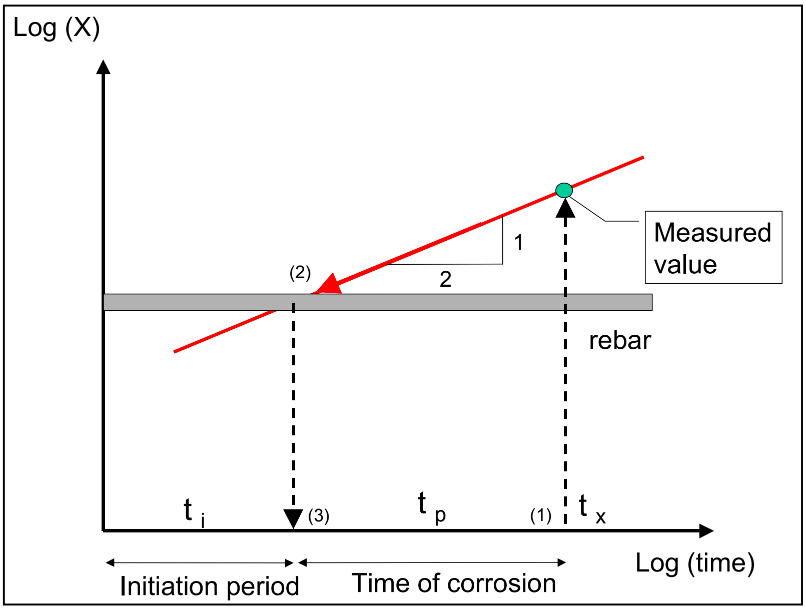


Figure 4.6. *Backextraplation to evaluate the time of corrosion*

Where $t_p = t_x - t_i$

t_x : age of the structure
t_i : initiation period

4.3.2 Determination of penetration P_X and the actual steel section

Actual penetration of corrosion may be calculated through two different methods:

- Simple measure of the residual diameter: This procedure may be only applied when the decreasing in steel section is appreciable (usually with corrosion due to chlorides)

  $P_x = (\phi_0 - \phi_t)/2$

where,

P_x the actual attack penetration measured by means of direct visual observation.
ϕ_0 the original diameter of the bar
ϕ_t is the diameter of the bar at time t

- Extrapolation with the use of *Representative Corrosion Rate* and the propagation time previously calculated as it is shown in Annex F, chapter F.2.1.

In order to achieve the residual cross section of steel, the type of corrosion (homogeneous or pitting) must be taken into account. Thus, the effective cross section can be calculated using the pitting factor α.

If geometrical measures can be made, the pitting factor α can be therefore obtained. If not, a expected value of ~10 may be used for pitting and 2 when carbonation.

The expressions to determine the residual steel section are included in Annex F, chapter F.2.1.

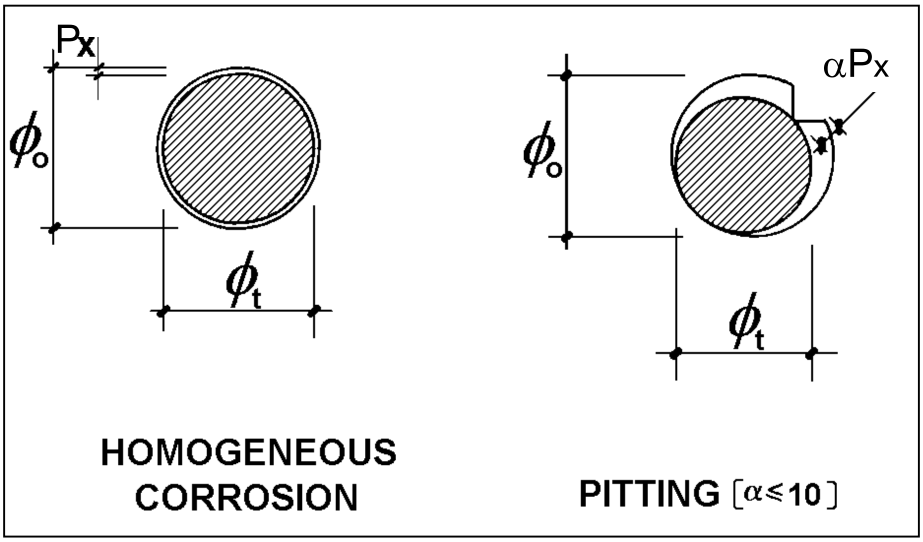


Figure 4.7. *Residual reinforcing bar section*

4.4. PROGNOSIS PHASE

Once present performance of the structure has been determined, next step is trying to predict how deterioration process will develop and when the structure will reach a non-acceptable performance level.

The inputs for prognosis phase will be:

- Present geometrical and mechanical characteristics of the assessed element.
- Aggressive progression characteristics (K_{CO2} , K_{Cl^-}, present depth and time since propagation period started)
- A representative value of I_{corr}^{rep} (see Annex C)

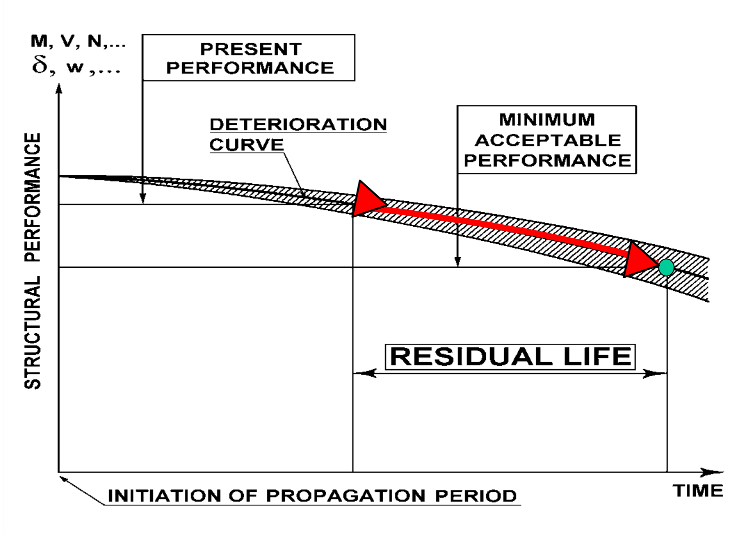


Figure 4.8. *Prognosis of the structural performance*

If the structure is actually in the initiation period, the result of the prognosis phase should be the time needed to achieve depassivation state, that is saying, the time needed for aggressive to reach the rebar. This value can be obtained again by applying the 'square root of time' model or someone similar as it is explained in point 4.4.1.

If the structure is corroding, the aim of prognosis phase is to determine when the structure will reach a previously determined minimum technical performance. The needed steps to achieve this goal are the following:

1. Define the minimum technical performance requested for the structure from ULS and SLS theory.

2. Determine the geometrical and mechanical characteristics that lead the element to reach this minimum threshold: rebars diameters, concrete cracking or spalling, etc.

3. Determine the attack penetration P_x that allows the condition referred above as it is explained in Annex F.

4. Assume an average value of representative I_{corr}^{rep} that could be considered for estimating future deterioration. For example, if the environment ensures a stable humidity content and temperature at rebar level, the measured I_{corr} can be used for prediction.

5. Determine the time needed to achieve P_x according to the representative value and the environmental conditions of the element. This value can be obtained from Annex F

4.4.1. Prediction of advance of aggressive

Prediction for future deterioration can be made with different levels of refinement. Only simpler ones will be described here.

Initiation period

One **simple** way to predict the rate of advance of carbonation front or of chloride threshold level, is by using the simple "square root of time" expressed in diagrams as that of Figure 4.9, where the vertical axis is log (depth of carbonation or of chloride threshold) and the horizontal axis is log of time [4.5].

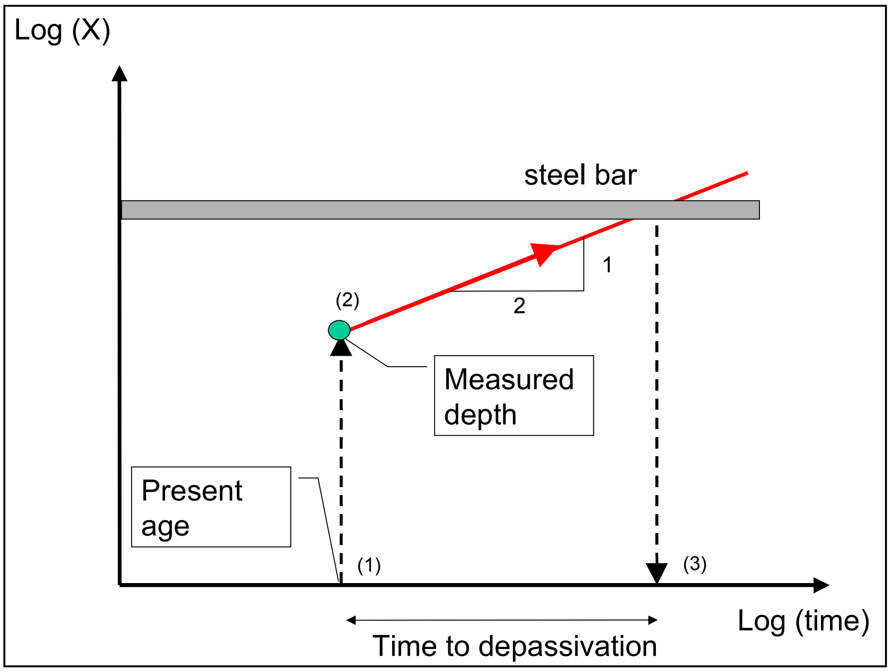


Figure 4.9 *Measured depth of carbonation (or of the chloride threshold) is the best description of the environmental influence and material characteristics. Prediction of future aging using actual values will increase the reliability of the diagnosis.*

Straigth lines with a slope of 0.5 in the log-log diagram are the mathematical description of carbonation or chloride penetration as a function of time for different concrete qualities in constant but different environments. A straigth line in the diagram passing the measured point will describe the carbonation front or chloride threshold progress, for concrete situated in that environment. This method will give conservative estimations due to the improved permeability properties with increasing depth. The concrete "skin" has such variations in porosity and humidity conditions that no existing theory could describe the first years of carbonation or chloride penetration. Therefore, minimum periods for accurate estimation of carbonation or chloride penetration rate with this method are after 2-4 years age.

There're also other more **refined** methods based on the more rigourous calculation of the diffusion processes involved on both carbonation or chloride penetration. They are reflected in Annex A.

It may also happen that other deterioration mechanism develop simultaneously. In these cases, the square root will not apply. Then, for those cases where exists frost attack carbonation will follow a linear progression. If exists cracks due to expansive reactions they may allow the entrance of chlorides into concrete depth.

Depassivation

Steel corrosion develops when either localized or generalized lowering of pH values occur. In the case of carbonation this pH decrease, is induced by cover neutralization. Values of around pH=8 induce the disappearance of the passive layer and a general corrosion starts.

Chlorides also induce a pH decrease, but localized. This decrease can be buffered by the surrounding alkalinity and therefore, the Cl^-/OH^- ratio is the important parameter to enable depassivation.

There is not a unique Cl^-/OH^- threshold value which could be generalized. It depends on the steel condition (rusted or rough surface, steel composition) as well as on the corrosion potential (level of oxygen). Depassivation was noticed in concrete for Cl^-/OH^- thresholds ranging between 2-8 (0.05% - 1% by concrete mass). An example on the influence of w/b ratio is given in Figure 4.10 [4.7] .

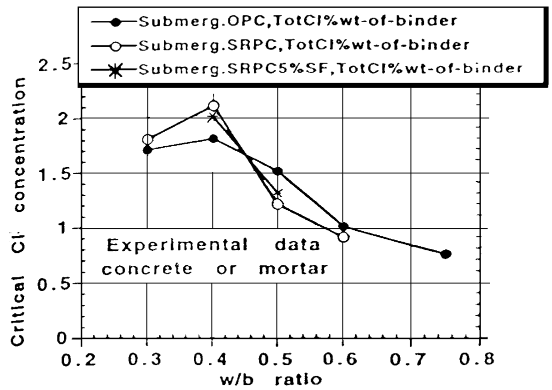


Figure 4.10 *Chloride threshold values in function of the w/b ratio.*

Therefore, chloride thresholds have to be identified, if possible, in each structure by analyzing the chloride contents near the rebar in the locations where corrosion is measured or recognized to have been initiated and compare them with those where steel remain passive.

Depassivation is not an instantaneous process, but it occurs along a certain time during which periods of activation - repassivation my happen. It can be identified when I_{corr}^{rep} overpasses the 0.1 $\mu A/cm^2$ during a certain period of time.

Attention has to be paid as well to other deterioration process developing as ASR or sulphate attack. Sulphates are also depasivating ions enhacing Cl^- effect, while alkalines may be beneficial by increasing the OH^- content.

4.4.2. Evolution with time of load-bearing capacity

The aim of the prognosis phase is to determine how the performance of the structure will evolve. Once the advance of aggressive has been predicted as it's shown in 4.4.1, the load-bearing capacity of the structure can be determined for each time of calculus. As it's reflected in figure 4.11, the procedure is so similar to diagnosis phase.

These steps can be followed for different time periods, so points of deterioration curve can be determined.

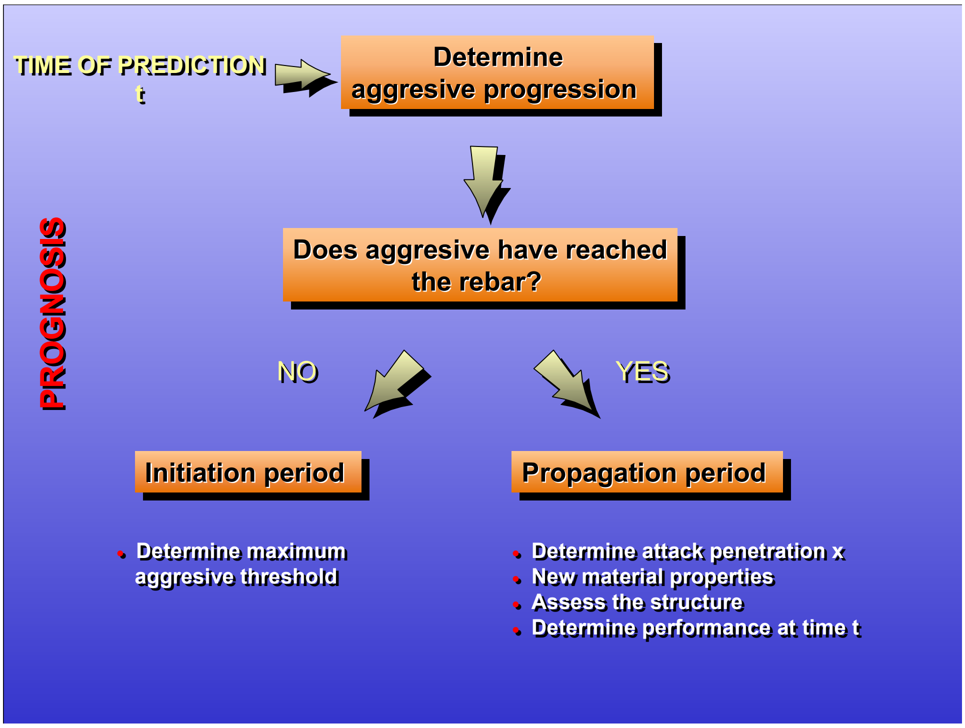


Figure 4.11 *Prognosis phase flow chart*

The verification of the structural performance at different values of t, at ULS and SLS can be carried out according to Annex F.

4.5. ACTIONS AFTER A DETAILED ASSESSMENT

Limits related to functional or serviceability criteria are widely variable because they finally depend on the cost for maintenance the owner is prepared to accept. For example, the basic functional criteria are clearly that the structure should be able to perform the basic function for which it was built. But there are other criteria, normally associated with functional requirements (maximum crack width, maximum deflection), which can be much more restrictive. The criterion established in the Conservation Strategy could be more or less flexible than that normally used for the design phase of a structure (e.g.: in the case of a structure affected by corrosion, should action be taken as soon as any crack width reaches the maximum level indicated by codes? Or should the intervention be decided when generalised spalling is taking place? Or should action be adopted even before any crack occurs?). In any case, these limits should be in accordance with the general philosophy inspiring the strategy selected.

In contrast to serviceability criteria, safety is not a matter of agreement with the client: Adequate safety is defined in the Building Codes or other legal provisions of each country. Some problems could arise because codes in many countries are not including the case of safety evaluation of an existing structure. And safety level to decide if an intervention is necessary or not can be established in a more or less restrictive way. Evaluation of an existing structure safety level is not necessarily governed by the same principles normally applied in design. Knowledge of certain aspects of the structure is better and this could be taken into account when assessing the structure. The Conservation Strategy should establish which will be the general principles to be applied to evaluate the safety level of the structure and the limits under which an intervention is needed.

After all, the selection of one specific strategy of conservation and the associated functional and safety criteria for a given structure will depend, obviously, in considerations related with economic reasons. It is clear that the essential main objective would be to minimise the total life cycle cost of the structure. But this basic goal will be affected by the type of structure considered, its importance and the environmental conditions affecting it. Accordingly, nothing general can be said establishing whether one of those conservation strategies will be better than the other for a specific structure.

If the design values of the effect of the actions, are lower than the resistance of the structure (or its elements) at Ultimate Limite States, estimated as it is said in annex F, no actions are required until the estimated time when the effects of the actions become higher than the structure resistance is reached.

If the design values of the actions are 10% higher than the structure resistance, a reassessment in a short time is needed (< 1 year).

If the design values of the actions are more than 10% higher than the structure resistance, an urgent repair work has to be considered.

Finally, if the allowable limits for the Serviceability Limit States are exceeded, the decission on the actions to be carried out shall consider the owner opinion.

Table 4.3. Actions after a detailed assessment

		Action to be taken
ULS	**R vs. S**	
	R > S	No actions are required until the estimated time when the effects of the actions become higher than the structure resistance is reached
	R<S<1.1R	Reassessment within 1 year
	S > 1.1R	urgent repair work
SLS		Agreement with owner's requirements

4.6. ASSESSMENT REPORT

With all data collected through the detailed inspection and tests carried out, a report would be prepared containing the following information:

1) Description of the structure.

2) Selection of the groups (lots) of elements.

3) Description of characteristic damages observed for each group of elements: crack pattern, delamination, spalling, ...as a result of inspection

4) Results obtained from the *in situ* test work

5) Diagnosis report which will contain:

    5.1 The causes of damage, and identification of the deterioration mechanism, attending to the environmental aggressivity.

5.2 The present situation of the structure: whether it is in the initiation period or in the propagation one (depth of aggressive front X_{CO2}, X_{Cl} and rate of advance V_{CO2}, V_{Cl^-}). This will include the plot of the $logX - log\ t$ diagrams.

5.3 Measured values of the I_{corr} of the reinforcements, (if measured) establishing its risk level (following Table ...) and its Representative value. Complementary values of resistivity and corrosion potential values should be also included.

5.4 The calculations of the present structural load-bearing capacity.

6) Prognosis report which will give the expected future evolution of the damage

6.1 If in the initiation period, through the use of the *log X-log t* diagram, prediction to the time of depassivation and expected onset of corrosion.

6.2 If in the propagation period:

- Propagation time T_P
- Calculation of the *Representative* I_{corr}^{rep}, progression of cracks and loss of residual steel section expected.

7) Structural model and load used in calculation of the actual load bearing capacity and future evolution.

8) The final aspects to be considered in the assessment Report are the Recommendations for:

8.1 - Frequency of further inspections
8.2 - Urgency of intervention (repair methods)

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ANNEX A. FUNDAMENTALS OF CORROSION

A.1 The corrosion process

A.1.1 The nature of the corrosion process

The mechanism of corrosion in aqueous media is of electrochemical nature. This means that the oxidation of the metal is counterbalanced by the reduction of another substance in another region of the metallic surface. Therefore, zones (anodes and cathodes) with different electrochemical potential, develop.

In the anodic regions the metal is oxidized and in the cathodic regions oxygen is the reduced species in alkaline and neutral media, as Figure A.1 shows:

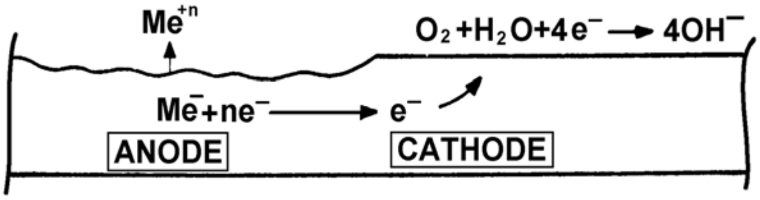


Figure A.1 *Simplified mechanism of aqueous corrosion: differentiation of zones in the metallic surface.*

The corrosion process mainly proceeds by the formation of numerous microcells in the corroding areas as Figure A.2 shows. In addition to the microcell activity and in the case of localized corrosion, where passive areas coexist with corroding ones, macrocell effects may develop. The galvanic current in the macrocell action is usually smaller than 10-20% the activity of the microcells, that is, of the total corrosion current.

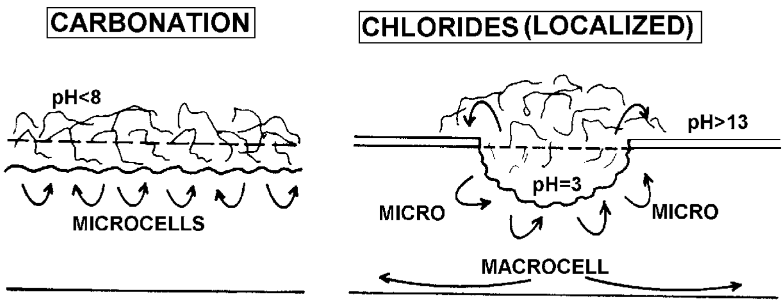


Figure A.2 *The corrosion process develops mainly by microcell action (short distance between anodes and cathodes). In the case of localized attack (right part of the figure) a macrocell action is superimposed. Its importance depends on the resistivity of the concrete surrounding the rebar.*

In the case of concrete the electrolyte is constituted by the pore solution, which is very alkaline (Figure A.3). This pore solution is formed by mainly a mixture of KOH and NaOH presenting pH values ranging between 12.6-14. The solution is saturated in $Ca(OH)_2$. Steel embedded in concrete is naturally protected by this high alkalinity and by the barrier effect of the cover itself

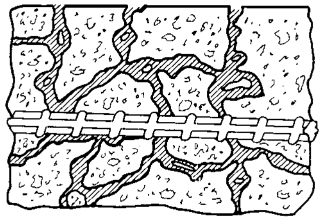


Figure A.3 *Pore network due to the excess of mixing water. Pores in the range of capilars are the important paths for the penetration of aggressives.*

The two main causes of corrosion are carbonation and the presence of chlorides[2.1] (figure A.4). Carbonation usually induces a generalized corrosion while chloride will lead into pitting or localized attack. The corrosion can be easily recognized by the rust presence on the rebar and by the appearance of cracks running parallel to the rebars.

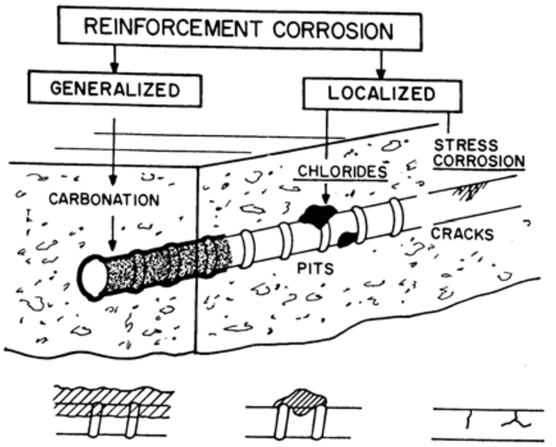


Figure A.4 *Types and morphology of the corrosion in concrete: generalized (carbonation), localized (chlorides) and stress corrosion cracking (in prestressing wires).*

Stress corrosion cracking of prestressing steel is a particular type of localized attack that will be not delt with in present Manual. It may develop in prestressed tendons.

A.1.2 Carbonation

Atmospheric carbon dioxide reacts with the calcium and alkaline hydroxides and cement phases, leading in a lowering of the pore solution pH value until values near neutrality. This process aims into the depassivation of the steel in contact with the carbonated zones.

Carbonation is a diffusion process and therefore, its depth progresses by an attenuation along the time. The modelling of carbonation is generally made by means of the simplified expression: $X=V_{CO2}\sqrt{t}$, where x is the carbonation depth, t is the time and V_{CO2} is the carbonation factor of the particular concrete. It does not develop if the concrete is water saturated or in very dry conditions. However, as cycling wet-dry periods are the usual environmental out-door conditions, the carbonation front can advance relatively fast[2.2].

As the corrosion is generalized, cracks (see Figure A.5) will appear running parallel to the rebars. Usually they appear not before 20 years life for a cover of 20-25 mm, what means that the corrosion rate are in general low. Spalling will be produced at later stages (see Figure A.6).

Figure A.5 *Crack pattern following the rebar network. It is due to the carbonation of the concrete cover.*
Figure A.6 *Spalling due to generalized corrosion.*

A.1.2.1 Rate of advance calculation

Several methods has been developed for the calculation of the carbonation front, most of them are based on the diffusion mathematics, although several modifications has been added to them in some cases.

a) Square root method.

For either carbonation and chloride advance, the simple square root law can be applied for the calculation:

$$X = V\sqrt{t} \qquad \text{[A1]}$$

where x is the penetration depth of the carbonation or chloride threshold, V is the velocity of advance and t the time. The law can be plotted in a log a log graph as shown in figure C4 where the dotted lines with slopes 0.5 represent the different velocities.

Providing the predefined chloride threshold is considered as shown in figure A5, this law can be easily applied to obtain a V_{CO2} and V_{Cl} which can characterize the quality of the concrete cover, by ploting in figure C4 the point corresponding to the present situation of the structure as is indicated in figure C6.

This figure also indicates how to find out in a graphic way the remaining time to depassivation: by plotting a line with slope 0.5 through the actual point and extrapolating to reach the rebar.

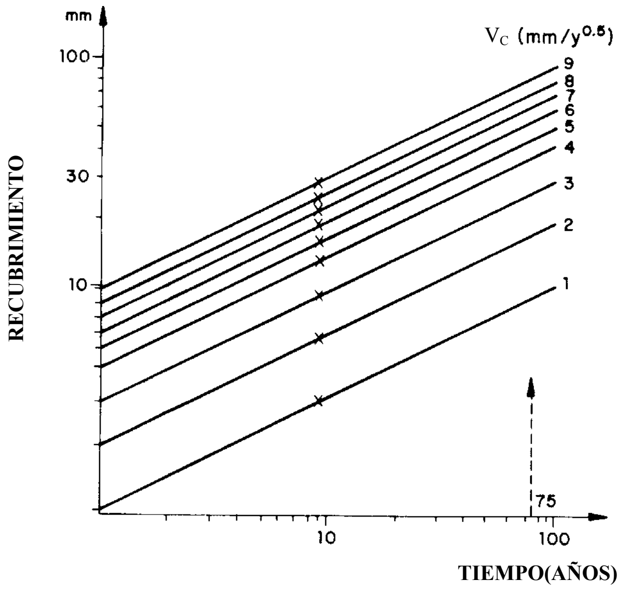


Figure A.7 *Square root model, strait lines with 0,5 slope in a log – log plot.*

More complex modelling can be also made. Next several methods will be summarized. All carbonation models are based on giving a value to the CO_2 diffusivity and to the external concentration of this gas. The effect of moisture is considered differently in the different models: some reduce the D_{CO2} when increasing RH, while other make the calculation of the wetness time and apply the occurrence of carbonation only when the concrete is dry enough.

b) Tuutti's model

Tuutti proposed a model for carbonation based on the moving boundaries diffusion theory. The expression results:

$$\frac{\Delta C_S}{\Delta a} = \sqrt{\pi}\left(\frac{k}{2\sqrt{D_{CO_2}}}\right) e^{\left(\frac{k^2}{4D_{CO2}}\right)} erf\left(\frac{k}{2\sqrt{D_{CO_2}}}\right) \qquad [A2]$$

where $\Delta a = c\,\frac{C}{100}\,DH\,\frac{M_{CO_2}}{M_{CaO}}$ and $k = \frac{\sqrt{t}}{X_{CO_2}}$

Where:

X_{CO2} is the depth of carbonation front at time t [m],
K is the carbonation rate in [m/s^{0,5}]
D_{CO2} is the effective diffusion coefficient for CO_2 [m^2/s]
Δa is the difference between concentration of bound CO_2 at the discontinuity and the unpenetrated concrete (kg CO_2/m^3]
ΔC_S is the difference between carbon dioxide concentration in the air and at the carbonation front [kgCO_2/m^3]
c is the cement content in [kg/m^3]
C is the CaO content in cement in %
DH is the degree of hydration of concrete,

M respective molar masses in [g/mol]
t is the age in [s]

The diffusion coefficient of a particular concrete is obtained through w/b ratio as indicated by figure A8 and has to be corrected in function of the relative humidity with the aid of figure A9.

The degree of hydration can be achieved through the table C1 and the external concentration of CO_2 may be supposed to be of 0,03 – 0,04 % in rural conditions and 0,10 % in urban conditions.

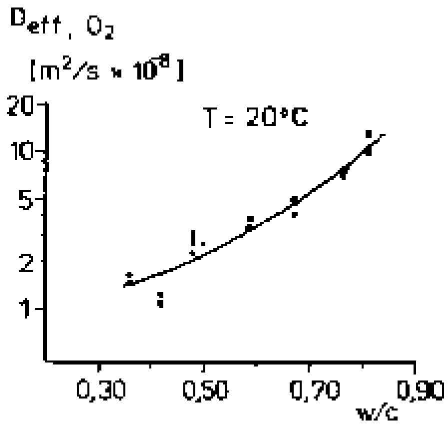


Figure A8.- Diffusion coefficient for O_2 and w/b ratio

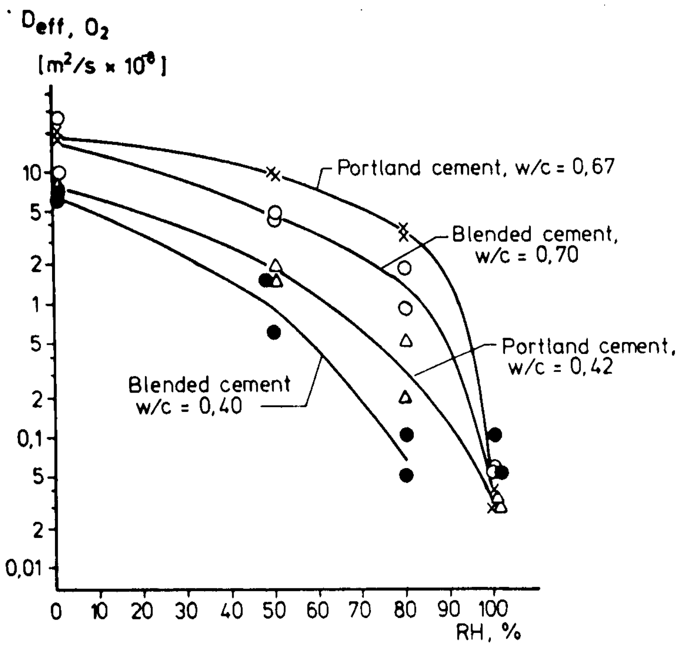


Figure A9.- Effect of RH in diffusion coefficient

W/b ratio	DH(%)
0,4	60
0,6	70
0,8	80

Table A1.- Degree of hydration and w/b ratio

c) Model of Bakker

Bakker's model is based in assuming diffusion in steady state conditions and the fact that concrete only carbonates when it is dry (wetness period).

In Bakker's model, the penetration depth X_{CO2} of the carbonation is given by the following expression:

$$X_C = A \sum_{i=1}^{n} \sqrt{t_{di} - \left[\frac{x_{ci-1}}{B} \right]^2} \qquad \text{(A3)}$$

A and B are functions which define the rate of carbonation and drying rate respectively:

$$A = \sqrt{\frac{2D_{CO_2}(C_1 - C_2)}{a}} \qquad \text{(A4)}$$

$$B = \sqrt{\frac{2D_V(C_3 - C_4)}{b}} \qquad \text{(A5)}$$

In which b may be calculated from,

$$b = w - 0{,}25cDH - 0{,}15cDHD_{gel} - wDHD_{cap} \qquad \text{(A6)}$$

The parameters are defined as follows:

D_{CO2} is the effective carbon dioxide coefficient in [m^2/s]
$C_1 - C_2$ is the carbon dioxide concentration difference between air and the carbonation front (kg CO_2/m^3)
a is the amount of alkalines in the concrete in [kg CO_2/m^3]
$C_3 - C_4$ is the moisture difference between the air and the evaporation front in [kg CO_2/m^3]
B is the amount of water to evaporate from the concrete in [kg CO_2/m^3]
DH is the hydration degree of concrete.
D_{gel} is the degree of physically bound water in the gel pores.
D_{cap} is the degree of physically bound water in the capillary pores.
T_{di} is the average length of the i_{th} drying period [s].
X_{ci-1} IS the carbonation depth after the (i-1)th wetting cycle [m]
C is the cement content in [kg/m^3].

d) CEB model

The model developed by CEB TG V Group is very similar to Bakker's one and is mathematically represented as follows:

$$X_C = \sqrt{\frac{2\,K_1\,K_2\,D_{CO2}\,C_S}{a}}\sqrt{t}\left(\frac{t_0}{t}\right)^n \qquad \text{(A7)}$$

in which,

$$a = 0{,}75\; C\; c\; DH\; \frac{M_{CO_2}}{M_{CaO}} \qquad \text{(A8)}$$

where:

X_C is the depth of carbonation front at time t [m],
D_{CO2} is the effective diffusion coefficient for CO_2 [m^2/s]
a is the difference between concentration of bound CO_2 at the discontinuity and the unpenetrated concrete (kg CO_2/m^3]
C_S is the difference between carbon dioxide concentration in the air and at the carbonation front [$kgCO_2/m^3$]
c is the cement content in [kg/m^3]
C is the CaO content in cement in %
DH is the degree of hydration of concrete,
M respective molar masses in [g/mol]
t is the age in [s]
K_1 is a constant parameter which considers the influence of execution in D_{CO2}
K_2 is a constant parameter which considers the influence of environment in D_{CO2}
n is a constant parameter which considers the influence of environment in time

Values of K_1 and K_2 may be obtained from table A2.

Exposure class	**Couring**	$\boldsymbol{K_1 x K_2}$
Indoors	Good	1,0
Indoors	Poor	2,0
Outdoors	Good	0,5

Table A2.- Proposed values for K_1 and K_2

Values of n are proposed in table A3.

Exposure class	$\boldsymbol{K_1 x K_2}$
Indoor conditions	0,0
Outdoors sheltered	0,1
Outdoors unsheltered	0,4

Table A3.- Proposed values for n

e) Parrott model

The parrot model is based on the empirical equation (A9) in which the carbonation front is calculated depending on the relative humidity in the concrete.

$$X_C = \frac{a\, k^{0,4}\, t_i^n}{c^{0,5}} \qquad \text{(A9)}$$

Donde,

$K = m \cdot k_{60}$

$$m = \begin{cases} 1.6 - 0.00115r - 0.0001475r^2 & r > 60\% \\ 1.0 & r \leq 60\% \end{cases}$$

$n = 0.02536 + 0.01785\ r - 0.0001623\ r^2$.

X_C is the penetration of the carbonation front in [mm]
k is the air – permeability of concrete cover depending on the relative humidity.
r is the relative humidity in the concrete (%).
c is the CaO content in the hydrated cement matrix of the covercrete in g/m^3
n empirical exponent
m empirical factor
a regression factor that can be assigned a value of 64 to be on the safe side.
t_i time of exposure in years.

A.1.3 Chloride attack

The chloride ions may be present in the concrete if they are added in the mix (admixtures, water or aggregates). However, this is fortunately not common. The most frequent is that chlorides penetrate from outside, either due to the structure is placed in marine environments or because deicing salts are used.

Figure A.10 *Localised attack due to chlorides.*

Figure A.11 *A pit due to chlorides in a prestressing wire.*

Chlorides induce local disruption of steel passive layer dealing into pits or localized attack. Some examples are given in figures A.10 and A.11.

Depending on how extended or localized is the corrosion, cracks may appear or not. In submerged zones sometimes the rebar corrodes without any external sign of cover cracking.

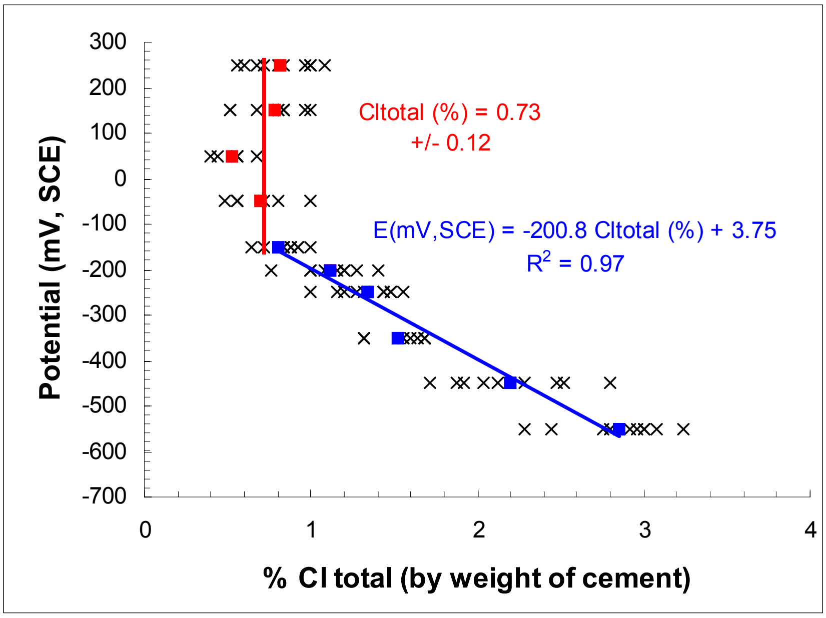
In submerged zones or in fully saturated concrete, chlorides penetrate by diffusion. However, in aerial zones or when submitted to cycles (deicing salts), capillary absorption may be a faster mechanism of penetration. In both cases, the penetration is as well dependent of the square root of time. Therefore, its modelling may be made similarly to the carbonation, by means of the simplified expression $x = V_{Cl^-} \sqrt{t}$.

Concerning the amount of chlorides needed to induce the onset of corrosion (threshold value), it depends on several factors. The factors influencing the chloride threshold are:

- Type of cement: finess, amount of C_3A, amount of gypsum, blending materials.
- Water/cement ratio (porosity).
- Curing and compaction (porosity).
- Moisture content and variation.
- Type of steel and surface roughness and condition (pre-rusted or not).
- Oxygen availability (corrosion potential when arriving the chlorides).

This multiple dependence makes difficult to fix a single unic value. However, a biunivocal relation appears when the concentration of chlorides is plotted versus the electrical potential. Thus, figure A.12 depicts this relation. The potential depends on the previously listed factors and therefore the same concrete will present different chloride thresholds depending upon the potential exhibited along the life. Dry concrete will promote more noble potentials while moist or submerged one will exhibit more cathodic ones.

Figure A.12 *Relationship between potential and total Cl^- threshold (in % by cement weight)*



In spite of the difficulty of fixing a reliable and general chloride threshold, all codes limit the chloride content in the mixing water. In absence of other value, this amount (in general 0.4% of cement weight) can be taken as reference. The minimum averaged chloride threshold (potentials more noble than -200mV SCE) derived from figure A.12 has resulted to be 0,7 % as total chloride per weight of cement.

A.1.3.1 Rate of advance calculation

a) Diffusion model

The basic assumption for chloride penetration is that these ions move in a semi-infinite medium being constant the external concentration. Although these two assumptions are not usually fulfilled, the expression resulting and given in expression C9 is worldwide used:

$$C(x,t) = C_i + (C_{sa} - C_i)\left[1 - erf\left(\frac{x}{2\sqrt{D_{Cl}t}}\right)\right] \quad \text{(A10)}$$

Where:

C(x,t) is the chloride content at depth x from the concrete surface, reached after a time t
D_{cl} is the achieved chloride diffusion coefficient of the concrete.
C_s is the achieved surface content (this is a best fit and not a true value).
T is the exposure period.
C_i is the initial chloride content at depth x.
Erf(ξ) is the error function.

b) Skin effect

It is oftenly found that the chloride profile recorded in real structures does not fit very well with the theoretically given in equation A10.

One particular case is shown in figure A13 where a maximum is detected far beyond the concrete surface. The reasons for that behaviour can vary, being one the carbonation of the concrete surface. It is known that carbonated cement phases do not bind chlorides and exhibit a higher chloride diffusion coefficient, therefore the chlorides move quickly towards the interior as carbonation or neutralization of concrete cover progresses.

The diffusion coefficient that really represents concrete bulk behaviour is that of the inner part of the profile (D_2). It can be calculated in two manners: a) by fitting the profile in the diffusion theory expression modelling two parallel media with different D (or skin effect calculation) and given by expression A11.

$$C_1(x,t) = C_S \sum_{n=0}^{\infty} \alpha^n \left(erfc\left[\frac{2ne+x}{2\sqrt{D_1 t}}\right] - \alpha erfc\left[\frac{(2n+2)e-x}{2\sqrt{D_1 t}}\right]\right) \quad \text{(A11)}$$

$$C_2(x,t) = \frac{2kC_S}{k+1} \sum_{n=0}^{\infty} \alpha^n \left(erfc\left[\frac{(2n-1)e+k(x-e)}{2\sqrt{D_1 t}}\right]\right) \quad \text{(A12)}$$

$$k = \sqrt{\frac{D_1}{D_2}} \qquad \alpha = \frac{1-k}{1+k} \quad \text{(A13)}$$

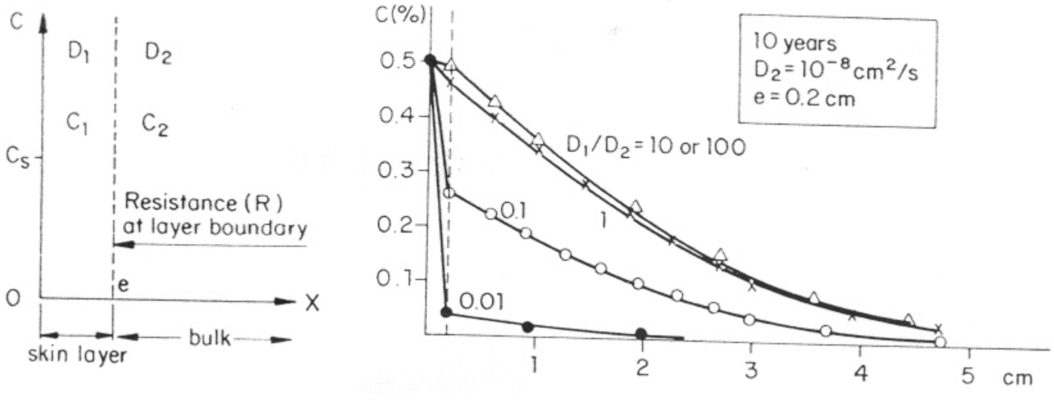


Figure A13 Skin effect.

If a resistance is going to be included into the interface of both materials, the new solution for C_2 is (A14).

$$C_2(x,t) = \frac{2kC_S R}{k+1}\sum_{n=0}^{\infty}\alpha^n\left(erfc\left[\frac{(2n-1)e+k(x-e)}{2\sqrt{D_1 t}}\right]\right) \qquad \text{(A14)}$$

Other solution may be a fitting the classical expression C10 into the inner part of the profile as shown in figure C10.

c) Age dependent diffusion coefficient

Either in carbonation or chloride penetration, it has been noticed a decrease of the D with time being the causes out of the scope of present document.

A useful way to represent the achieved diffusion coefficient through time may be:

$$D_{Cl}(t) = D_{Cl}(t_0)\left[\frac{t}{t_0}\right]^n \qquad \text{(A15)}$$

which has to be substituted in any of the expressions previously given.

A 1.4. Stress corrosion cracking

Stress corrosion cracking, SCC, is a particular case of localized corrosion. It occurs only in prestressing wires, because its development is produced when wires of high yield point are stressed to a certain level and are in contact with a specific aggressive medium.

The process starts by the nucleation of microcracks in the surface of the steel (figure A11). One of them may progress to a certain depth, from which the crack velocity is very high and the wire breaks in a brittle manner in relatively short time.

The mechanism of nucleation and, mainly of progression of SCC in still subjected to controversy. Nucleation can start in a surface unhomogeneity, rest of rust or in a pit. The progression in enhanced by the generation of atomic hidrogen in the bottom of the crack. From the several mechanisms proposed to explain the process that based in the concept of "surface mobility" seems the best fitting experimental results. The surface mobility assumes that the progression of the crack is not of electrochemical nature but it is due to the mobility of atomic vacancies in the interface metal/electrolyte.

The only manner to diagnose the occurrence of SCC is by the microscopic examination of fractured surfaces in order to identify the brittle fracture. Thus, figure A.15 shows a ductile fracture of a prestressing wire and figure A.16 a brittle one (no striction is produced). Figure A.17 shows a photograph of a wire which has suffered SSC in a chloride contaminated concrete.

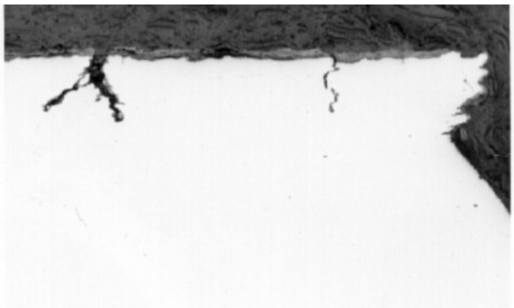
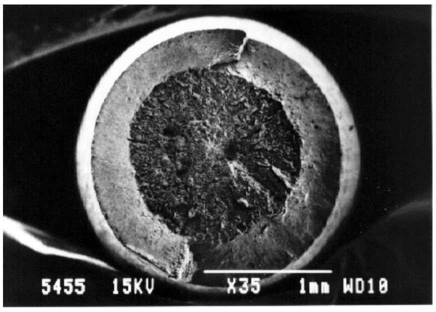


Figure A.14 *Microcracks generarated from the metallic surface*



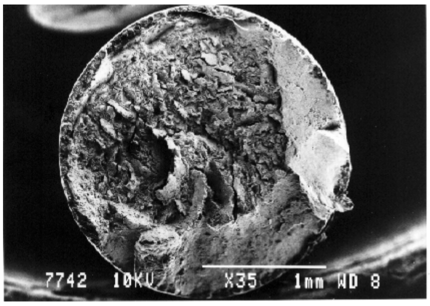


Figure A15 *Ductile fracture*

Figure A.16 *Brittle fracture*

Figure A.17 *SCC in a prestressing wire*

In the SCC phenomenon the metallographic nature and treatment of the steel plays a crucial role. Thus, quenched and tempered steels are very sensitive while the susceptibility of cold drawn steels is much lower. The use of the former are forbidden for prestressing in many countries. More information about SSC is given in Annex D.

A.3 Service life

In the case of reinforcement corrosion, the most simple and descriptive model for service life is due to Tuutti and is shown in Figure A.18.

This well known model considers:

- An initiation period, which consists of the time from the erection of the structure until the aggressive agent (either chlorides or the carbonation front) reaches the rebar and depassivates the steel.

- A propagation period from the steel depassivation until a certain unacceptable level of deterioration is developed in the structure.

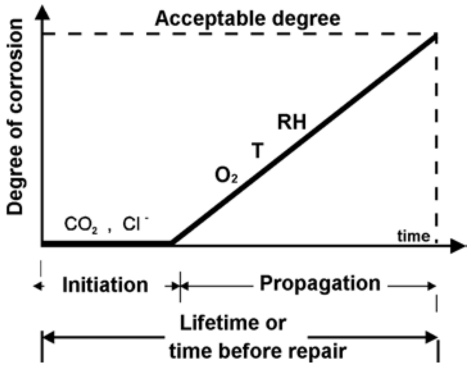


Figure A.18 *Service life model for reinforcement corrosion.*

A.2.2 Corrosion: structural effects

As figure A.19 is indicated that the first direct effect of the corrosion on a steel element is its section decrease due to the corroding process. Iron oxides (rust) resulting from the corrosion process have a larger volume than the original steel, in the case of reinforced concrete structures, and this effect induces internal stresses in the concrete which may lead to cracking or even spalling of concrete cover.

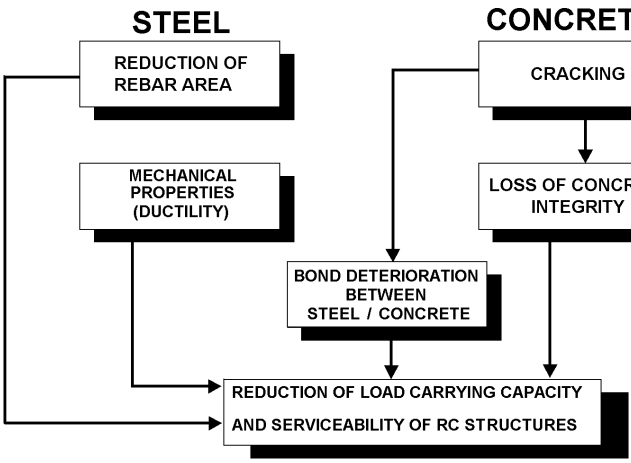


Figure A.19 *Consequences of the corrosion in the structural performance.*

Corrosion also may reduce the steel elongation at maximum load, affecting subsequently the structure ductility. On the other hand, the composite action of concrete and steel in a reinforced concrete structure is based on the bond between them. And this is also affected by corrosion through several mechanisms:

a) Increasing of hoop stresses due to pressure of rust, producing concrete cracking,
b) Change of properties of the interface concrete-steel.
c) Corrosion of stirrups.

Accordingly, reduction of structural capacity of reinforced concrete elements affected by rebar corrosion is mainly due to the following three main phenomena, which are direct consequence of corrosion:

- Reduction of rebar section due to corrosion
- Reduction of bond strength
- Loss of concrete integrity due to cover cracking and/or spalling

The rate of developing of these phenomena is function of different parameters as corrosion current (I_{corr}), type of aggressive, environmental moisture, time since propagation period was initiated, and reinforcement or structural detailing.

Therefore, the following factors need to be considered for the structural assessment:

Environmental conditions

Nature of corrosion (generalized or localised)

Corrosion rate

Reinforcement detailing

Consequences of failure

Structural redundancy

The reinforcement or structural detailing does not only have a direct influence on those basic phenomena, but the potential reduction of structural capacity for a given level of corrosion damage is function of factors like structural redundancy or anchorage of main reinforcement.

A.3. Environment

A.3.1. Effect of environment on rebar corrosion

The main environmental influencing parameter on the corrosion is the moisture content of the concrete which is dependent on the environmental humidity. The concrete moisture content will influence the electrical resistivity and the oxygen availability at the rebar level.

As Figure A.20 shows, when the pores are fully saturated with moisture, the resistivity reaches its lower value, but oxygen access is restricted as it has to dissolve in the pore water. In consequence, the corrosion rate may be lowered by an oxygen diffusion control.

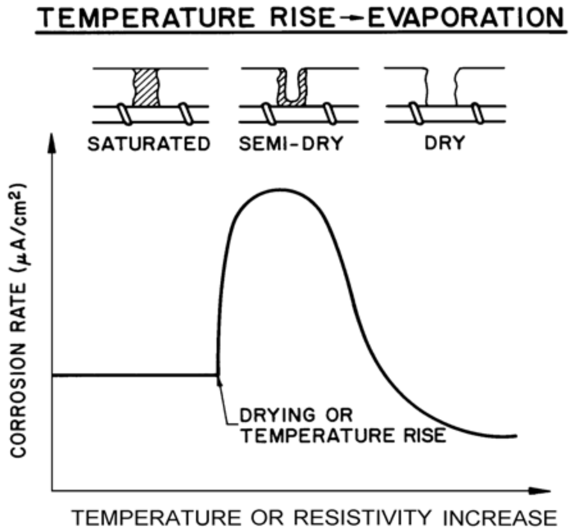


Figure A.20 Evolution of corrosion rate with the variations of moisture in the concrete pores. Influence of temperature in this moisture content.

When the pores start to dry, the oxygen can easily reach the rebar and the corrosion will increase accordingly. This is illustrated in the middle part of Figure A.20. However, when the concrete dries (right part of the Figure A.20), the resistivity increases and the corrosion will be slowed down again.

In consequence, the conditions for reaching a maximum in the corrosion rate is in nearly saturation, when both oxygen availability and resistivity counterbalance their effects (the middle stage in Figure A.20).

Temperature will influence as well the corrosion process. It produces also two opposite effects: to accelerate or retard the reaction. This is indicated as well in Figure A.20. When temperature rises, evaporation of pore water is induced and oxygen is removed from the pore solution. Therefore, although the corrosion process is stimulated by the rise in temperature, this may be counterbalanced by the increase in resistivity (evaporation) and the removal of oxygen (smaller solubility at higher temperatures). An opposite effect is induced by a lowering of temperature in semi-dry concrete as condensation is induced.

All this means that the effect of daily and seasonal variations of RH and T on the corrosion of rebars, cannot easily be quantified.

2.3.1.1 Outer and inner concrete environment

All these effects are not homogeneous in the concrete mass, but a gradient of moisture is produced from the concrete surface to the interior. Figure A.21 depicts an example of this gradient made evident by calculation. It will depend on the rebar position (cover depth) that the climatic cycling will influence the corrosion process.

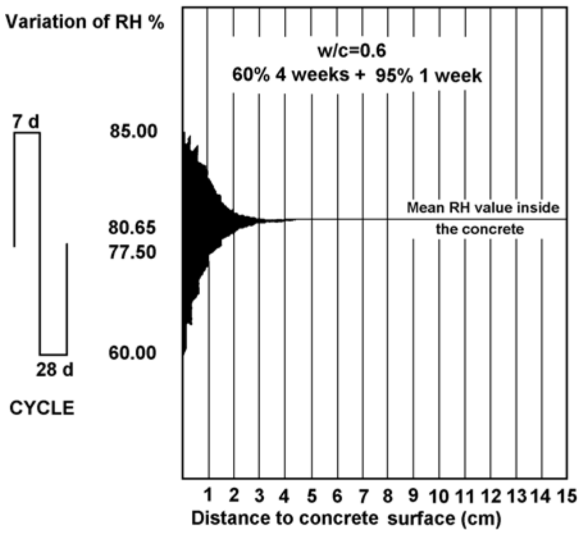


Figure A.21 *Computed example of moisture content in the concrete cover of a external moisture cycle of 7 days at 95% of RH and 4 weeks at 60% RH.*

In consequence, it has to be stressed the difference between the outer and the inner concrete environment. It is the inner one which will influence the corrosion process. The inner temperature is usually quite similar to the outer one at the rebar level. However the RH may be very different. Figure A.22 shows the variation along 2 years of recording of external and internal RH in a concrete having 3% $CaCl_2$ added in the mix.

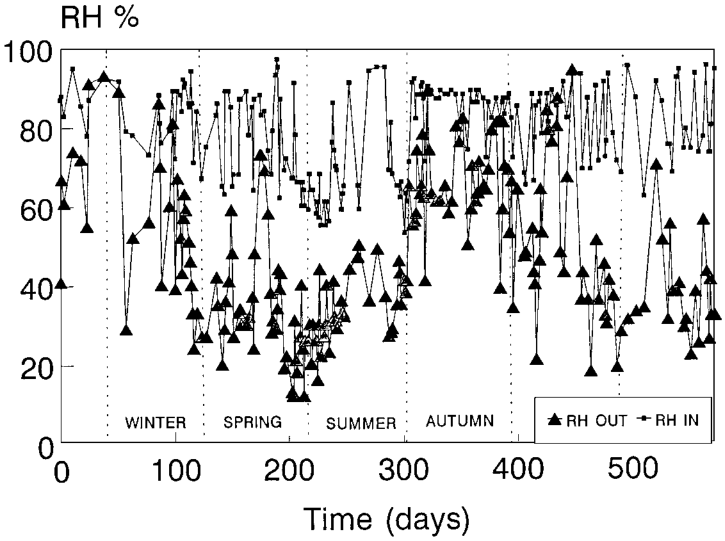


Figure A.22 *Evolution of concrete inner RH, in a concrete with 3% of $CaCl_2$ added in the mix, and environmental RH (outdoor Madrid climate non sheltered from rain). The RH inside concrete presents smaller changes than the externel RH in the atmosphere.*

The difference between inner and outer environment does not apply in cracks wider than 0.3 mm, where it is more similar to the outer one. Cracks usually are a shorter path for reaching the rebar (shorter initiation period), however during the propagation period as they may dry out quicker, only in some particular cases (submerged structures) they may accelerate the corrosion process.

An example of how the corrosion parameters envolve in the concrete element whose inner environment is shown in Figure A.18 is depicted by Figures A.23 (E_{corr}) and A.24 (I_{corr}).

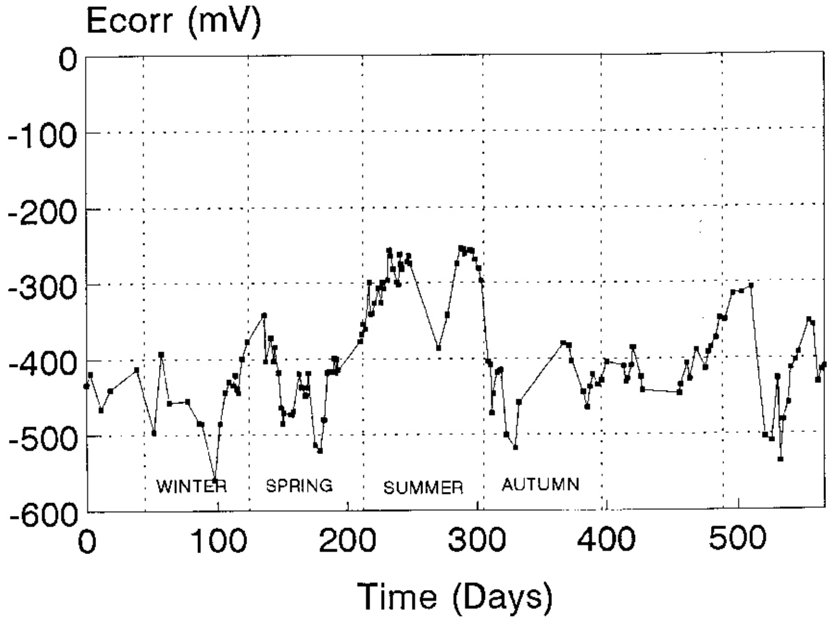


Figure A.23 *Evolution of corrosion potential along time for a rebar embedded in the concrete of the previous figure.*

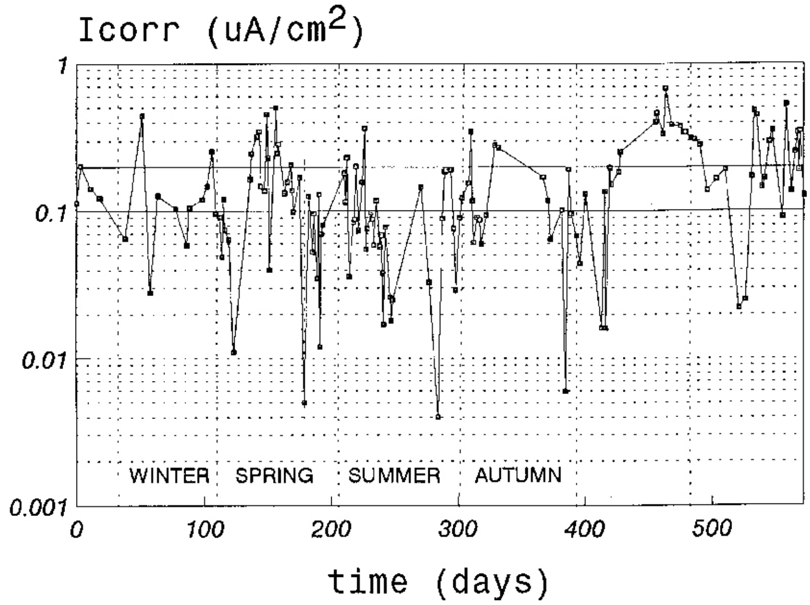


Figure A.24 *Evolution of I_{corr} in the same concrete.*

The scatter of I_{corr} values recorded are due to the opposite influence of moisture and temperature evolution during the day/night cycles (Figure A.24). The corrosion process cannot reach a steady-state situation due to the wide range of values and the rate of variation of concrete inner moisture. Similar trend of I_{corr} evolution happens in the corrosion of steel when directly exposed to the atmosphere. The concrete acts as a sponge, retaining longer the moisture.

These circumstances make necessary to define a Representative I_{corr} value which could average the yearly behaviour. A methodology is proposed in Annex E.

ANNEX B. ENVIRONMENTAL CLASSIFICATION

B.1 INTRODUCTION

The aspects of environmental that have to be well identified during a preliminary inspection, in order to be able later to classify the exposure class:

a) Whether the chlorides are present or not. There may be three sources of chlorides:

   1. Added in the mix. (in the cases of old buildings and works before 1975 or in zones where pure water is not available or clean aggregates are scarce).

   2. Added externally as de – icing salts or being present in chemical contact with the concrete (industrial plants, swimming pools, etc.)

   3. Marine environment.

b) In the case of presence of chlorides, the distance of concrete surface to the source of chlorides, which also means to the source of moisture.

c) In the absence of chlorides (carbonation will be then the type of aggressive), the regime of moisture in contact with the concrete is the main factor to be identified. At this respect the concrete can be:

   - In dry indoor conditions (interior of heated buildings).

   - Sheltered from rain (interior of not heated buildings and outdoor exposure).

   - Non – sheltered from rain and therefore subject to cycles of wet – dry conditions.

   - Permanent wet in contact with a source of moisture.

B.2 EN206 CLASSIFICATION

Present EN206 establishes several classes according to general provisions previously exposed. Thus, table b .1 shows the environmental classification proposed in EN206, and actually in use in Europe.

Class designation	Description of environment	Informative examples where exposure classes may occur
1.- No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there's free/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2.- Corrosion induced by carbonation		

Where concrete containing reinforcement or other embedded metal is exposed to air moisture, the exposure shall be classified as follows: Note: the moisture condition relates to that in the concrete cover to reinforcement or other embedded metal but, in many cases conditions in the concrete cover can be taken as reflecting that in the surrounding environment. In these cases classification of the surrounding environment may be adequate. This may not be the case if there is a barrier between the concrete and its environment.		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
1.1.1.1 XC 3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3.- Corrosion induced by chlorides other than from sea water		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car parks
4.- Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures

ANNEX C.- CALCULATION OF A REPRESENTATIVE CORROSION RATE, I_{corr}^{REP} VALUE

C.1. Corrosion rate definition

The corrosion rate of a metal is defined as the metal loss per unit of surface and time.

$$CR = \frac{metal\ loss}{surface \cdot time} \qquad [C1]$$

The units of expressing the corrosion rate are diverse. One representing the above equation is $\left[\frac{g}{cm^2 \cdot year}\right]$, but the most usual ways are the following two:

a) As attack penetration depth (see figure C1), either uniform or localized, expressed in **μm/year** or mm/year, which is calculated from the metal mass loss through the metal density in order to have the P_x in $\mu m(10^{-6}\ m)$ or mm.

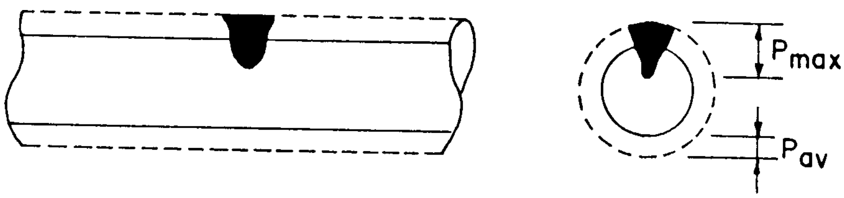


Figure C.1. Averaged (P_{av}) and maximum pit depth (P_{max}) or maximum attack penetration

b) In $\mu A/cm^2$ by using Faraday's law which converts the mass units in electrical units. Faraday's law is:

$$\frac{I \cdot t}{F} = \frac{\Delta w}{W_m / z} \qquad [C2]$$

where I= electrical current in Amperes, t= time in seconds, F= Faraday's constant (96500 coulombs), Δw= mass or weight loss in grams, W_m= molecular weight of the metal and Z= valence exchanged.

The equivalence for the steel between both ways of expressing the corrosion rate is

$$1\mu A/cm^2 <> 11{,}6\mu m/year$$

C.2. Measurement of the corrosion rate

It can be made through gravimetric measurements, which in real size structures is only feasible if a piece is cut from the main rebar and initial weight is obtained from steel density. Therefore the attack penetration is obtained by means of measuring the diameter loss and/or the electrical parameters.

C.2.1. Measurement diameter loss

The loss in bar diameter can be measured through the use of a calibre device providing the rust is previously removed in order to get a clear steel surface.

C.2.2. Electrical techniques: Polarization Resistance method

From the several methods to measure the electrical parameters related to the corrosion process, the so called Polarization Resistance method, is the most widely used to determine the corrosion rate. The method is based imposing a small electrical current ΔI or voltage, ΔE, to the metal in contact with the electrolyte, and measuring the corresponding response in voltage or current:

That is:

$$R_p = (\Delta E / \Delta I)_{\Delta E \cong 0} \qquad [C3]$$

The instantaneous corrosion rate is obtained from the following expression:

$$I_{corr} = \frac{B}{R_p} \qquad [C4]$$

where B is a constant that for in-situ tests is usually taken to be 26mV.

As the measurement is non-destructive towards the metal and takes only few seconds or minutes to perform a measurement, it can be repeated indefinitively without perturbing the process. In consequence, the periodic measurement of Rp enables the monitoring of all kind of corrosion processes and has been used in many systems metal electrolyte.

C.3. Measurement of Rp in large structures

Direct estimation of true R_p values from ΔE/ΔI measurements is usually unfeasible in large concrete structures. This is because the applied electric signal tends to vanish with distance from the counter electrode, CE rather than spread uniformly across the working electrode, WE as shown in figure C.2. Therefore, the action of the electric signal cannot be related to any specific area.

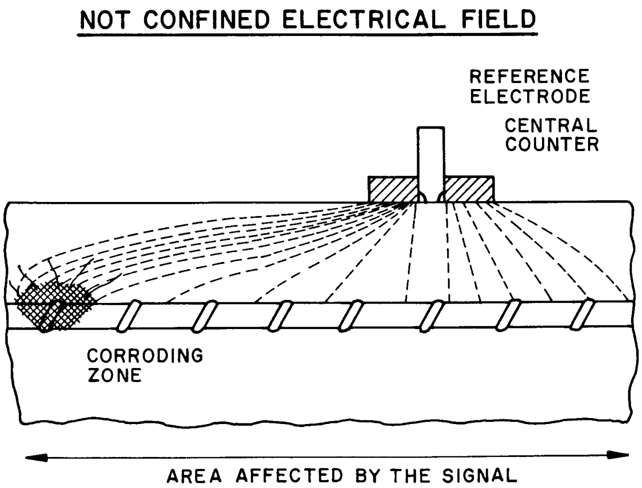


Figure C.2. The electrical signal applied from a small counter electrode tends to vanish with the distance. The active spots have a higher drainage ability.

Hence, $\Delta E/\Delta I$ measurements on large structures using a small counter electrode provides an apparent polarization resistance (R_p^{app}) that differs from the true R_p value depending on the experimental conditions. Thus, if the metal is actively corroding, the current applied from a small CE located on the concrete surface is 'drained' very efficiently by the rebars and it tends to confine itself on a small surface area. Conversely, if the metal is passive and R_p is high, the current applied tends to spread far away (e.g., around 50 cm) from the application point. Therefore, the apparent R_p approaches the true R_p for actively corroding reinforcement. But when the steel is passive, the large distance reached by the current necessitates a quantitative treatment.

There are several ways of accounting for a true R_p value, among which the most extended one is the use of a guard ring, in order to confine the current in a particular rebar area, as figure E.3 depicts. However, not all guarded techniques are efficient. Only that using a “Modulated Confinement” controlled by two small sensors placed between the central auxiliary electrode and the ring, is able to efficiently confine the current within a predetermined area.

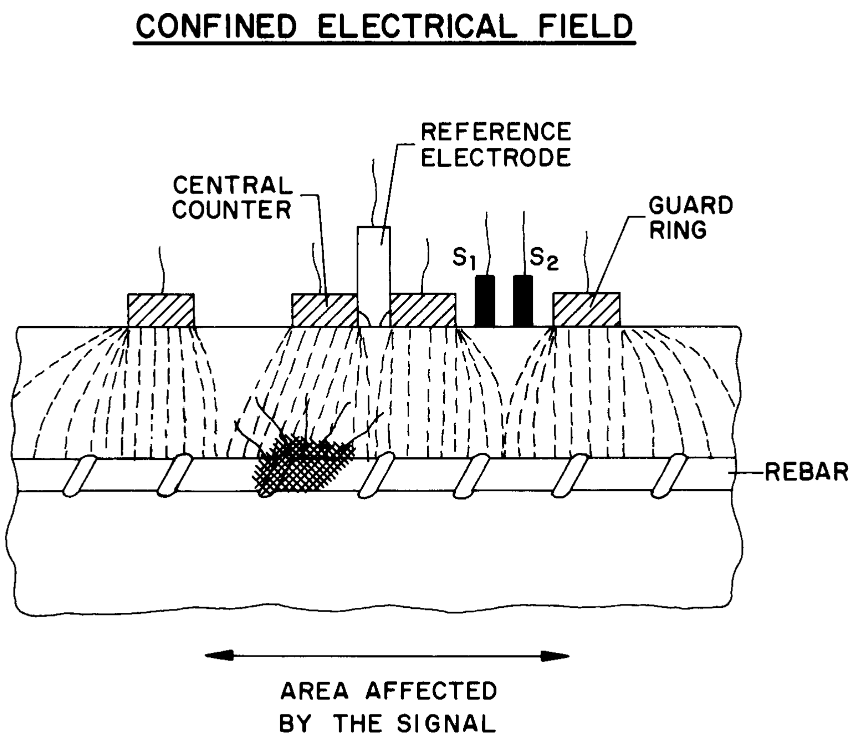


Figure C.3. Principle of the modulated confinement of the current by means of two sensors (ring control) placed between the central auxillary electrode and the ring.

Other methods such as the direct measurement of the so called critical length, L_{crit}, (the distance reached by the current) or minimising the error, by using large auxiliary electrodes, present the inconvenience of being unable to detect localized corrosion, as was shown in figure C2. Due to the draining effect of the active spots on the current, they will take most of the current when applied within the area of influence of the L_{crit}. Thus, although the R_p so obtained could be correct, the localization of the corroding spots is uncertain.

The measurement of the value of $\Delta E/\Delta I$ in large structures without any mathematical correction or confinement (called apparent R_p) does give erroneous values of the corrosion rate. This is the case when using pulse techniques or galvanostatic pulse with which gross errors can be obtained.

C.4. Ranges of corrosion rate measured in situ

The experience on real structures has confirmed the ranges of values previously recorded in laboratory experiments. In general, values of corrosion rates higher than 1μA/cm^2 are seldom measured while values between 0.1-1μA/cm^2 are the most frequent. When the steel is passive very low values (smaller than 0.05-0.1μA/cm^2) are recorded. The I_{corr} values found in the laboratory and on real size structures on-site are ranked in the levels shown in Table C1.

Table C1. Levels of corrosion rates measured in laboratory and on-site.

Corrosion rate (μA/cm^2)	**Corrosion level**
< 0.1	Negligible
0.1 - 0.5	low
0.5 - 1	moderate
> 1	high

A comparison of on-site I_{corr} values to electrical resistivity has allowed the authors to also rank the resistivity values, as described in Table C2. The values apply to either OPC or blended cements.

Table C2. Levels of concrete resistivity regarding risk of corrosion

Resistivity (kΩ.cm)	**Corrosion risk**
>100-200	- Negligible corrosion, concrete too dry
50 – 100	- Low corrosion-rate.
10 – 50	- Moderate to high corrosion when steel is active.
<10	- Resistivity is not the controlling parameter of the corrosion rate.

C.4.1. Influence of climatic parameters

The most challenging aspect of on-site measurement, is the fact that the electrochemical parameters are weather dependent and, therefore, its actual value will depend on the particular climatic conditions around the structure. When a single value of the corrosion current is measured on-site, it may happen that the concrete is dry at the time of measurement and therefore, mislead the deduction of its corrosion state. Next a methodology is proposed for obtaining a Representative on-site I_{corr}. Two main alternatives exist: be a) to take several readings during a certain period of time, or b) making a single isolated measurement.

Several measurements

The optimum would be to take at least four measurements over 12-month period to take into account the different weather seasons. Thus, the following extreme climatic conditions should be considered for the sake of taking measurements:

- Dry periods with low temperatures
- Period of low temperatures after raining continuously during at least one or two days.
- Dry periods and high temperatures.
- Periods at high temperatures one week after raining continuously during two or three days.

Measurements during these periods will enable the recording of nearly minimum and maximum values of the corrosion current. A representative value can be obtained by averaging the values recorded:

$$I_{corr}^{\mathrm{Re}\,p} = \sum_{0}^{n} \frac{I_{corr}(t)}{n} \qquad \text{[C5]}$$

Single measurements

When isolated measurements are the only possibility, obtaining representative I_{corr} is more uncertain. In order to interpret the readings in the most accurate way, the procedure recommended, is based in the relation between resistivity and I_{corr} (figure E.4). Figure C4 indicates that in the same structure the usual relation between I_{corr} and ρ when plotted in a *log – log* diagram is linear with a slope of -1 (I_{CORR} - 3 10^4 /ρ). In consequence, the procedure proposed is the following:

- After having measured the corrosion current, cores should be taken close to the measurement points. Cores are returned to the laboratory. Then they are conditioned to a moisture of 85% RH (for structures sheltered from rain) or vacuum water saturated, (for non sheltered or submerged ones) when the cores are equilibrated to the moisture their minimum electrical resistivity, ρ_{min} is measured.
-
- Finally, the values of the I_{corr} - ρ registered on-site are plotted in the graph (points A) (Figure C.5). The extrapolation to the maximum I_{corr} is made to reach the ρ_{min} (point B) at 85% HR or core saturation.

Finally the I_{corr}^{rep} will be calculated averaging both values, the single value $I_{corr,\ single}$ and the maximum value achieved at laboratory $I_{corr,\ max}$. [C6]

$$I^{rep}{}_{corr} = \frac{I^{sing}{}_{corr} + I^{max}{}_{corr}}{2} \qquad \text{[C6]}$$

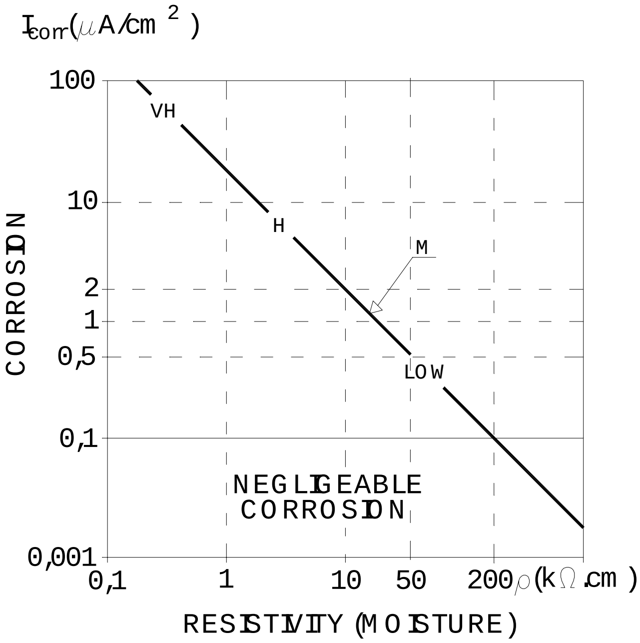


Figure C.4. General averaged relation between corrosion rate I_{corr} and resistivity ρ

Figure C.5. Graph I_{corr}-ρ where VH- Very high, H- high, M-Moderate, L- Low, A: Measurement points, B: Extrapolation to minimum ρ, C: Maximum expected I_{corr}.

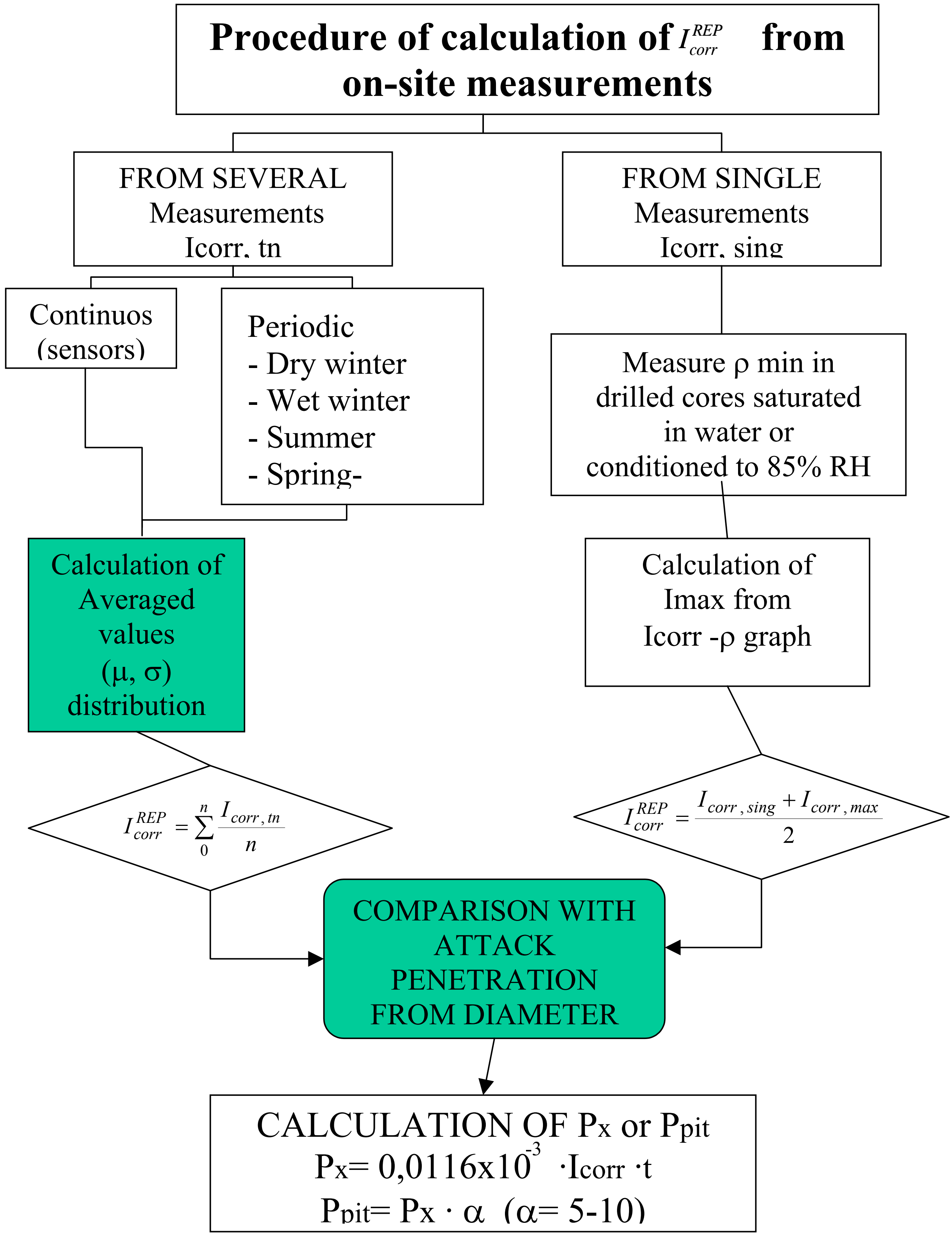

Procedure of calculation of I_{corr}^{REP} from on-site measurements
FROM SEVERAL Measurements Icorr, tn
FROM SINGLE Measurements Icorr, sing
Continuos (sensors)
Periodic
- Dry winter
- Wet winter
- Summer
- Spring-
Measure ρ min in drilled cores saturated in water or conditioned to 85% RH
Calculation of Averaged values (μ, σ) distribution
Calculation of Imax from Icorr -ρ graph
$I_{corr}^{REP} = \sum_{0}^{n} \frac{I_{corr,tn}}{n}$
$I_{corr}^{REP} = \frac{I_{corr,sing} + I_{corr,max}}{2}$
COMPARISON WITH ATTACK PENETRATION FROM DIAMETER
CALCULATION OF Px or Ppit
$P_x = 0{,}0116 \times 10^{-3} \cdot I_{corr} \cdot t$
$P_{pit} = P_x \cdot \alpha \ (\alpha = 5\text{-}10)$

Figure C.6.

C.4.2. Relation between E_{corr} and I_{corr}

Although in a single structure, there may be a relation between E_{corr} and I_{corr}, the relation cannot be generalized. Thus figure C.7 shows the case of multiple testing points showing the big scatter found between E_{corr} and I_{corr}.

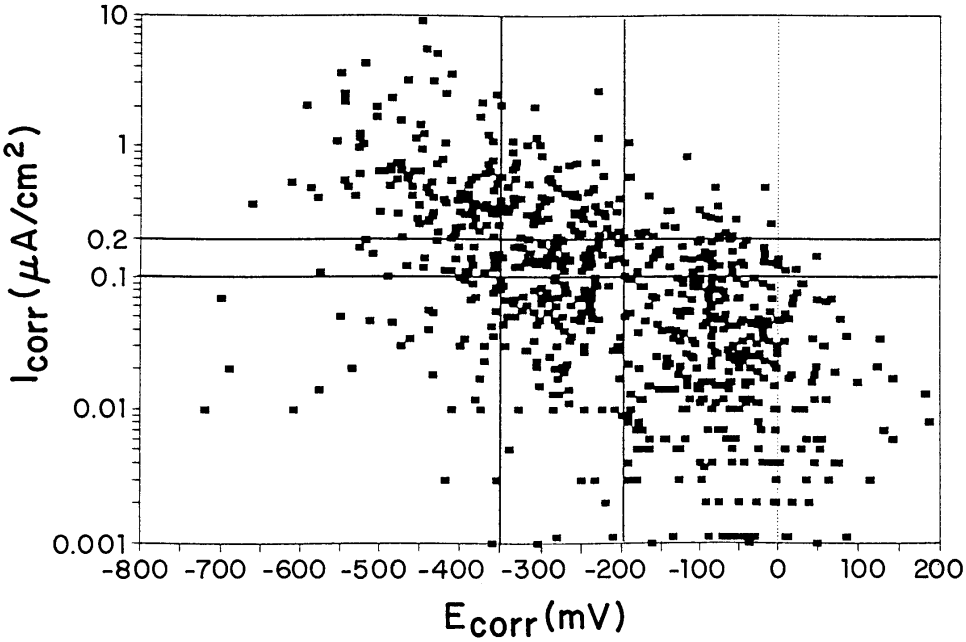


Figure C.7. Lot of E_{corr} versus I_{CORR} values in numerous structures.

C.5. Values into Calculation of loss in bar cross section from I_{corr}^{REP} values

The loss in cross section can be obtained from corrosion measurements either obtained electrochemically or by direct measure in the steel bar previous removing of rust. The attack penetration P_x is defined as the loss in cross radius as shown in Figure E.8. In the case of electrochemical measurements P_x is obtained through the expression:

$$P_x(mm/y) = 0.0116 \cdot I_{corr}^{REP} \cdot t_p \quad [C7]$$

being t_p= the time in years after corrosion started and 0.0116 a conversion factor of $\mu A/cm^2$ into mm/year (for the steel). This expression implies the need to know when the corrosion has started in order to account for t_p.

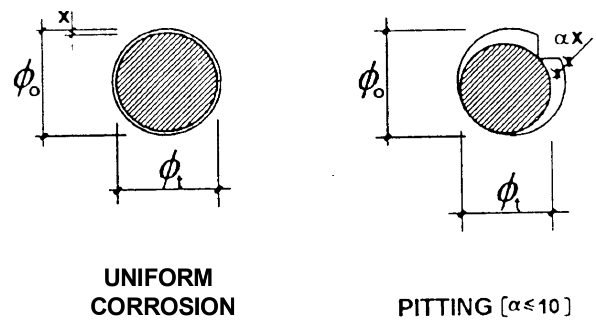


Figure C.8. Residual steel section loss considered for the cases of uniform and localized corrosion.

When the corrosion is localized (right part of figure 4), the maximum pit depth is calculated by multiplying expression [C7] by a factor named α which usually takes a value of 10. Hence expression [C7] above becomes,

$$P_{pit} = 0.0116 \cdot I_{corr}^{REP} \cdot t_P \cdot \alpha \qquad [C8]$$

C.5.1 Calculation of the length of the propagation period

The calculation of the t_p can be made if the carbonation front is known or the chloride profile is recorded. By extrapolating back the rate of carbonation or chloride penetration, the time when corrosion started can be approximated (assuming a certain chloride threshold value) by finding out when these fronts reached the rebar. This is shown in figure C.9, where the penetration depth of the aggressive front is plotted versus the log of the time. The back extrapolation of the line of slope equal to 0.5 passing by the point of the front depth gives the time when this front reached the bar.

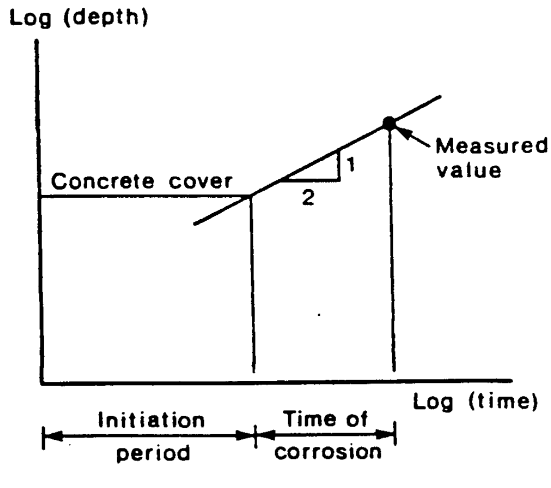


Figure C.9. When the carbonation or chloride threshold front is beyond the rebar, it can also inform on the time when depassivation was produced. This back extrapolation allows the evaluation of the time of corrosion (propagation period).

C.6. Representative I_{corr} values in the lack of measurements

When any kind of direct measurement can be performed on-site (neither the loss in cross diameter or the instantaneous I_{corr}), it can be used as I_{corr}^{REP} the mean values given in table C3 according to exposure classes of EN-206.

Table C3. Ranges of I_{corr} values suggested for exposure classes of EN206.

Exposure Classes		I_{CORR} [µA/cm²]	
0	No risk of corrosion	~ 0.01	
Carbonation		1.1.1.2 Partially carbonated	*Totally carbonated*
C1	Dry	~ 0.01	~ 0.01
C2	Wet – rarely – Dry	0.1 – 0.5	0.2 – 0.5
C3	Moderate humidity	0.05 – 0.1	0.1 – 0.2
C4	Cyclic wet – dry	0.01 – 0.2	0.2 – 0.5
Chloride initiated corrosion			
D1	Moderate humidity	0,1 – 0,2	
D2	Wet – rarely – dry	0,1 – 0,5	
D3	Cyclic wet – dry	0,5 – 5	
S1	Airborne sea water	0,5 – 5	
S2	Submerged	0,1 – 1,0	
S3	Tidal zone	1 – 10	

ANNEX D. TEST DESCRIPTIONS

D.1. CRACK MEASUREMENT

Purpose

Rust products of corrosion have a larger volume than the steel that replaces in the rebar. This change in volume generates a tensional state into concrete that leads to its cracking.

Two periods can be considered when studying the deterioration of the concrete integrity due to corrosion :

a) Cracking initiation period, where the cracks born, grow and reach the concrete surface
b) Cracking propagation period, where the crack width at concrete surface grows until cover spalling might occur.

Once cracks have reached the concrete surface, it is of the most interest not only to record all the cracks present in the concrete surface but also to monitor their growth and development along time and space.

Description

Once the cracks have reached the surface of the structure, they can be measured by a number of different methods. For a complete characterisation of cracks, it should be necessary to report for each one different aspects as its direction, depth and width.

Regarding the scope of measurement method, they can be classified in two main groups:

- Crack width measurements. The width of initial macro-cracks (shrinkage cracks, initial defects) can be measured by a crack-width gauge from the birth of the structure and continued measurements can show the changes in time. There are several devices that allow this measures as crack width gauges, measuring microscopes, suitable tell tales and adhesives, vernier callipers or Demec gauges.

- Crack mapping. To achieve an effective assessment of the structure, a systematic sequence of the inspection process must be planned. It is recommended to divide the whole structure in its different components to verify if the nature and kind of problem is the same in all elements or if more than one different problem exists.

  For each of these parts, the extent of cracking in a region can be assessed and fixed by video, photograph or paper. The obtained crack pattern can be recorded and compared with the former ones, so a monitoring of the crack evolution can be made.

Efficiency / accuracy

The efficiency and accuracy of crack measurement is directly related to the ability of detecting cracks. The crack width that can be detected depends on several factors as:

- The skill and experience of the operator
- Previous knowledge of the structure

- The state of the concrete surface.
- Accessibility. Obviously, it is not the same to work close the surveyed element than if devices as binoculars are needed to have visual access to the element.
- Environmental conditions. On a clean concrete surface which has been wet and is drying off, it is possible to locate very fine cracks by the visible lines and patches which follow the cracks and longer for a while after the rest of the structure has become dry. Strong slightly oblique lighting also helps to show up very fine cracks.
- Etc.

Usually, for a preliminary inspection, the crack width that could be noticed could vary from 0.4 to 0.5 mm. In the case of a detailed inspection, developed once a preliminary inspection has detected main damages, noticeable crack with diminishes to 0.2-0.3 mm. If special devices as are used, cracks of 0.1 mm width could be detected.

D.2. COVER THICKNESS

Purpose

Concrete cover acts as a physical barrier between the reinforcement steel and the environment the structure is exposed to. Depending on its characteristics, those agents that lead to corrosion processes will access to the embedded reinforcement by a faster or slower way. The likely type of damage will also be influenced by the cover thickness and its relationship with rebar diameter, varying from local spalling to delamination between bars.

For both inspection and repair works of a structure, it is necessary to know exactly not only the location and main characteristics of the reinforcement steel (number of bars, diameter, etc.) but also the thickness of the concrete cover.

The measurement of the cover thickness can be performed at two different ages of the structure. It can be carried out on new structures to assure that design specifications have been observed but also when corrosion is noticed. The variability of the cover thickness is a crucial parameter for a correct probabilistic analysis of the service life.

Description

The most common method of measurement of the cover thickness is the use of covermeters devices. Their operation is based on the different electromagnetic properties of the reinforcing steel, with regard to those of the surrounding concrete.

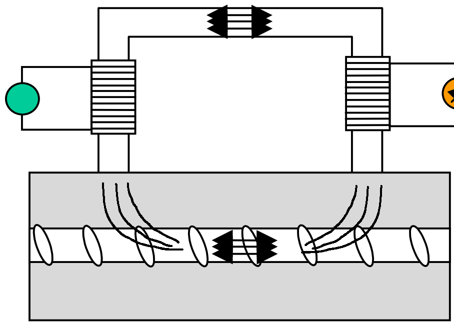
When an alternating magnetic field intersects an electrical circuit, an electrical potential is induced in that circuit, which is proportional to the rate of change of the magnetic flux through the area bounded by the circuit (Serway, 1983). This electromagnetic induction principle allows covermeters to measure the change in magnetic flux caused by the presence of steel.

There are two different types of covermeters: those based on the magnetic reluctance principle and those based on eddy currents.

Magnetic reluctance-based covermeters

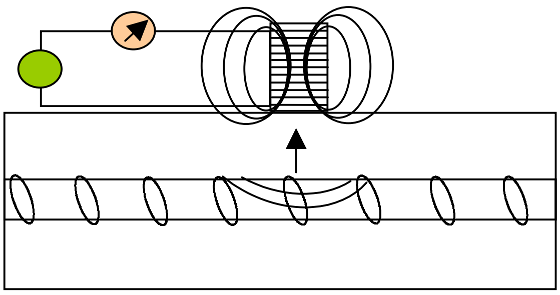
Based on Faraday's law, when an alternating electric current is applied through an electrical coil located around a ferromagnetic device, a magnetic field is induced so a magnetic flux flows from one pole to the other of the device. The resistance of the media to this flow is called reluctance. Magnetic reluctance-based covermeters detect the changes in the reluctance due to the presence or absence of a rebar. If it is not present, concrete reluctance is so high that magnetic flux flow is very small. By the other hand, electromagnetic properties of steel make reluctance very low, so magnetic flux flow increases. A sensing coil located in the device measures these changes.

To obtain the cover depth, a calibration relationship should be stablished to correlate and interpret the maximum deflection readings. Since the size of the bar affects the reluctance of the magnetic circuit, there would be a separate relationship for each bar size.



Eddy current-based covermeters

The presence of an electric current near an electrical conductor as a reinforcement bar induces both a magnetic field and so-called eddy currents through that element. These eddy currents generate also a secondary magnetic field that interacts with the first one.



Eddy current-based covermeter measures the changes produced in the applied electric current due to the interaction of the secondary magnetic field. This field generates an electric current, which, according to Lenz's law, opposes the primary one. The net current, considered as combination of primary and secondary ones results lower than the applied current, is measured by the covermeter's sensoring device. So changes can be analysed to obtain the reinforcement location or the cover depth.

It is important to point out that there are other available methods for locating embedded reinforcement steel as ground penetration radar or x-ray.

Efficiency / accuracy

There are several factors that influence the measurement of cover thickness and the location of the embedded reinforcement steel. They can be classified in two main groups: those factors affecting the measurement itself, and those regarding to the own accuracy of the used device.

Number of bars

As it was referred before, the covermeter obtains the cover depth by measuring the changes in the magnetic field generated by the presence of embedded rebar and relating this change to the area of the reinforcement affected by the magnetic field.

If there are several bars located in parallel, when the covermeter device is moved parallel to the rebar axis, a minimum in the measured value should be obtained when the device passes just in the middle of the two bars. As closer are the bars, this minimum value will be higher so the difference between peaks and valleys in the measurement signal will be mitigated. This process can lead to an error of the measured value so ,cover can not be estimated correctly using the single-bar calibration relationships.

Presence of perpendicular bars

When the covermeter is located over a reinforcement bar, the presence of rebar located perpendicular to the axis of the device has less influence in the cover thickness measurement than parallel located ones. Anyway, measured values are affected by an error that should be corrected by means of the factors obtained from the calibration relationships of each device.

Presence of magnetic particles in the concrete

When concrete has ferromagnetic materials as pozzolans, oxides, fly ashes, etc. covermeter may register variations in the magnetic field or in the applied current without the presence of reinforcement steel. This aspect does not affect the detection of the embedded rebar, but obviously, a correction procedure should be applied in order to obtain a realistic cover thickness values. The method is as simple as considering the measured value in a known zone without reinforcement steel as base value for later measurements.

Although accuracy depends also on each commercialised device, British Standard 1881 Part 204: 1988 requires that when measuring cover to a single bar under laboratory conditions, the error in indicated cover should be no more than plus or minus 5 % or 2 mm whichever is the greater. For site conditions, an average accuracy of plus or minus 5 mm or 15 % is suggested as being realistic in the British Standard.

D.3. HALF-CELL POTENTIAL MEASUREMENT

Purpose

Corrosion of steel leads to the coexistence of passive and corroding areas in the same bar forming a short-circuited element with the corroding area as anode and the passive surface as cathode. The cell voltage in this macroelement induces a current flow across the concrete is coupled with an electric field. This electric field can be measured and represented as equipotential lines that allow the study of the state of a metal in its environment.

The main objective of potential measurements on a structure is to locate areas in which reinforcement has become depassivated and hence, is able to corrode if appropriate oxygen and moisture conditions occur. Furthermore, there are other purposes potential measurement may be used for:

- Locate and define those places where other kind of tests should be developed in order to get a more precise and cost-effective information about the state of the structure.

- Evaluate the efficiency of repair works by controlling the corrosion state of the rebar

- Design preventive measurements as cathodic protection or electrochemical restoration techniques.

Description

As referred before, the objective of potential measurement is to determine those areas of corroding reinforcement. To achieve this goal, first it is completely necessary to define a work strategy that provides a fast and economical overview on the state of the structure. This strategy must involve the definition of a co-ordinate system to correlate readings and measuring points. A grid usually makes it with a cell size that varies from 15 square centimetres to 2 square metres, depending on the type of the structure, its characteristics and the scope of the work. The size of this co-ordinate system is determining the accuracy of the measurements.

To measure the half-cell potentials on a structure, a good electrical connection to the reinforcement has to be made. It could be made by means of a compression-type ground clamp or by brazing or welding a protruding rod, but a direct contact should not be made if reinforcement steel is connected to an exposed steel member. The other input of the high impedance voltmeter must be the external reference electrode placed on the concrete surface by a wet sponge in order to provide a good electrolytic contact between them. The sponge should be always wetted with a diluted solution of detergent.

It is completely necessary to assure the electrical continuity of the reinforcement steel. Measuring the resistance between separated areas checks it. If resistance values are less or equal than 0.3 Ω, electrical continuity is indicated.

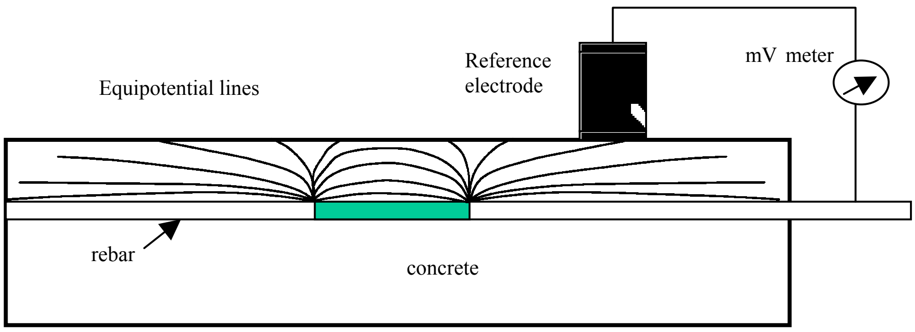


Fig. .Potential measurement of concrete reinforcement

Potential measurements can be performed with a single electrode or with one or several wheel electrodes. These ones facilitate the potential survey of large bridge decks, parking decks, etc. allowing up to 300 m^2 per hour when they are connected to microprocessor controlled data-loggers.

Once the data is obtained, the best way of its representation depends on its number and the type of the structure. So it varies from tables to a coloured grid map of the potential field, where every individual potential reading can be identified as a small cell and a contour line map can be obtained interpolating between point measurements with different algorithms.

In a grid map, a coloured cell that has associated its potential value represents each measurement point. The colour gradation step should not be greater than 50 mV in order to provide a clearest way to result interpretation. A 3-D surface can also be represented both by measured and interpolated values.

Of course, potential measurement can be represented by all standard statistical ways as cumulative frequency distribution or histograms. The kind and depth of the study to be developed should impose the design requirements for these graphics.

Efficiency/accuracy

The interpretation of the potential readings has evolved during last years. According to ASTM C 876-87 standard, a threshold potential value of –350 mV CSE was established. Lower values of potential suggested corrosion with 95 % probability; if potentials are more positive than -200 mV CSE, there is a greater than 90 % probability that no reinforcement steel corrosion occurs, and for those potentials between -200 mV and - 350 mV corrosion activity is uncertain. Later practical experiences have shown that different potential values indicate corrosion for different conditions so absolute values can not be taken into account to indicate corrosion hazard, that is, the relationship between concrete condition and potential values is not well-defined enough, with the exception of those potentials at extreme ends.

There are different aspects that must be considered in potential measurement.

Environmental factors

A wide range of factors influences the corrosion potentials as:

- Concrete moisture content. Depending on moisture condition of concrete, its resistivity varies. Changes in moisture content may lead to a difference of potentials up to 200 mV. It is important to consider not only different moisture conditions in a determined point but changes along the whole structure. Potential values become more negative as concrete moisture increases.

- Chloride content. Field experience on a large number of bridge decks has shown a certain correlation between concrete chloride content and the potential values. The most negative values coincide with the areas of high chloride content.

- Concrete carbonation. As carbonation process leads to an increase of concrete resistivity, potential measurements show more positive values on both passive and corroding rebars.

- Cover thickness. As concrete cover increases, the difference between corrosion and passive potential values diminishes, resulting on a uniform potential value at infinite. Thus, the location of small corrosion spots gets more difficult with increasing cover depth.

- Polarisation effects. The anode is polarising the passive rebars in the vicinity of the corroding area to negative potentials. The shift of potentials to more negative values is higher in low resistivity concrete than in high resistivity concrete. This lead to a better small corroding area detection in the first case, but not for high resistivity concrete due to its less polarised area of passive rebars.

- Oxygen content. Conditions of aeration, i.e. oxygen access, strongly determine rest potential values of passive steel in concrete. Low oxygen content leads to a pronounced decrease of the rest potential. In wet concrete due to very low oxygen diffusivity coefficient conditions, may arise in a shift of potential to comparably negative values so passive steel may show negative potentials similar to those of corroding steel. This leads to the risk that passive areas under low aeration conditions could be considered as corroding areas.

Regarding the uncertainties in the measurement itself, it is necessary to point out that the reference electrode does not measure a ‘true’ potential value of anode and cathode but a mixed one. It is due to concrete cover does not allow potentials to be measured directly above the rebar.

There are also another influencing factors like:

Type of reference electrodes used

The unpolarizable reference electrodes (Calomel or $Cu/CuSO_4$) are more accurate than other types (carbon, for instance). As ell their maintenance regime (contact membrane) is very crucial for their accuracy. Finally, temperature influences also the potential, being the unpolarizable electrodes the most reversible ones.

Copper/Copper sulphate electrode is the most used one for in-situ potential measurement, whereas calomel and silver chloride electrodes are used more in lab works.

Measurement procedure. Grid spacing

To determine the optimum grid spacing, it is necessary to determine the maximum distance from a corroding rebar at which there is no evidence of potential change. Measurements made with a big grid cell size could not detect corrosion activity whereas minimum spacing generally should provide high differences between readings. The spacing must be adequate to the type of structure surveyed and the expected use of measurements.

D.4. CORROSION RATE

Purpose

The measurement of the corrosion current gives the quantity of metal that goes into oxides by unit of reinforcement surface and time. The amount of oxides generated is directly linked to the cracking of concrete cover and the loss in steel/concrete bond, while the decrease in steel cross-area in addition significantly affects the load-bearing capacity of the structure.

The rate of corrosion is therefore an indication of the rate of decrease of the structural load-carrying capacity. The four main consequences of reinforcement corrosion are:

- The loss of section of steel and concrete
- The loss of ductility of the steel
- The loss in bond between steel and concrete

Apart from the calculation of the loss in rebar cross section the corrosion current may be used for the following purposes:

a) Identification of corroding zones. Corrosion maps enable to identify the corrosion zones in the same manner than the potential mapping.
b) Evaluation of the efficiency of repair techniques such as use if corrosion inhibitors, patching or realkalisation.

Description

The measurement of the corrosion current is made by means of a reference electrode, which indicates the electrical potential, and an auxiliary electrode, which gives the current. In on-site measurements, in addition to the central circular auxiliary electrode a second auxiliary electrode (guard ring) is being used in order to confine the current into a limited reinforcement surface (see figure). This guard ring has the objective to balance the electrical field produced by the central auxiliary electrode as figure F depicts.

The most used technique to measure corrosion current is the so-called polarisation resistance, Rp, which is based in very small polarisations around the corrosion potential.

$$R_p = \frac{\Delta E}{\Delta I} \qquad \Delta E < 20 mV$$

The corrosion current Icorr is inversely proportional to the Rp by means of the relation

$$I_{corr} = \frac{B}{R_p}$$

Where B is a constant which oftenly takes the value of 26 mV.

For on-site measurements, the location of the measurement points is a very crucial issue in order they were representative of the deterioration process. The location may be selecting by using a hypothetical grid with a fixed space although also can be selected the most affected points and compared with the least affected ones. In order to select these measurement points, also other complementary techniques such as the corrosion potential or the resistivity may be previously used.

Concerning the interpretation of the values of the corrosion current, table 1 gives the ranges linked to the loss of steel cross section

Icorr values (μm/year)	Corrosion level
< 1	Negligeable
1 - 5	Low
5 - 10	Moderate
> 10	High

Table 1. Relation between corrosion rate and level of corrosion

Efficiency / accuracy

With regard to the accuracy of these measurements, two main aspects have to be taken into account:

1.- The morphology of the corrosion, that is to say, how much localised is the corroding area. If pits are produced, the maximum pit depth can be calculated by means of a factor (α) which takes a value between 3 and 10. Thus,

Ppit = Icorr. α

Or to measure directly in the bars the maximum pit depth P_{pit}

2.- In on-site measurements it is necessary to take always into account the use of corrosion rate meters which could correctly appraise the dispersion of the current with the distance. That is, it is necessary to use the modulated confinement of the current (controlled guard ring) or the potential attenuation method. One of the two modes of operations have to be used, otherwise the error can be of one or two orders of magnitude.

Other influencing parameters related to the environment of the concrete itself are:

1) **Macrogalvanic effects**. The corrosion itself is based in a electrochemical cell function, however macrogalvanic effects are called there have several centimetres or decimetres separating a corroding zone from a no corroding one (the cathode). That is, a corroding area (by microcells) may be influenced by a passive region and develop a higher corrosion current than if isolated from this passive region.

2) **Chloride content**. The chlorides not only act by depassivating the steel, but also enhance the corrosion rate. As higher is their amount, higher is the corrosion until a certain limit, beyond which the corrosion current may be even decrease. This has been interpreted to be due to the saturation of the concrete pore solution and the removal of oxygen that a concentrated solution induces. The corrosion then decreases or even stops.

3) **Moisture content**. The moisture or liquid water in the concrete pores is the most relevant parameter influencing the corrosion current. It is responsible of the electrolyte continuity (pore connectivity) and of the oxygen availability at the steel surface. In addition, liquid water moisture fixes the concrete electrical resistivity, which is the most comprehensive parameter determining the corrosion current. Oxygen content is more secondary although below a certain amount where corrosion practically stops.

4) **Temperature**. It has an opposite effect on the corrosion rate. On one hand when the temperature increases, the moisture evaporation, which may counter-balance the trend to increase the rate by lowering the activation energy of the process and viceversa. Only in water saturated structures, the temperature may present the expected direct relationship with the corrosion rate.

D.5. CONCRETE RESISTIVITY

Purpose

The electrical resistivity of concrete is, together with available oxygen content, one of the most influencing material parameters concerning to corrosion intensity. Its measurement is increasingly being used in conjunction with potential mapping as a property that may be useful for monitoring and inspection of concrete structures to assess the severity of reinforcement corrosion problems.

The resistivity of a given structure provides information about the risk of early corrosion damage, because it's revealed that there is a linearly relationship between corrosion rate and electrolytical conductivity, that is, low resistivity is correlated to high corrosion rate.

It is necessary to point out that corrosion rate is not only controlled by concrete resistivity, so this parameter can not be considered as determining factor to define or prevent a potential damage to the structure and to establish the need for applying preventive or repair measures. Next, a brief evaluation scheme for concrete resistivity is shown:

Resistivity	*Corrosive potentiality*	*High moisture risk*	*Cl or CO_2*	*Corrosion*	*Action*
ρ ≥ 200 kΩ.cm	Low	No	-	Low	-
		Yes	No	Low	-
			Yes	Low-Medium	Monitoring
ρ > 50 kΩ.cm	Medium-Low	-	No	Low	-
			Yes	Low-Medium	Monitoring
ρ > 10 kΩ.cm	Medium	-	No	Low	-
			Yes	Medium-High	Monitoring
ρ < 10 kΩ.cm	High	-	No	Low-Medium	
			Yes	High	

Source. Manual de inspección, evaluación y diagnóstico de corrosión en estructuras de hormigón armado. DURAR. CYTED

Description

Although the objective of this report is to focus on inspection methods, which are mainly referred to on-site testing, a selected inspection strategy could also consider the opportunity to carry on laboratory tests. This is the reason why they are mentioned here.

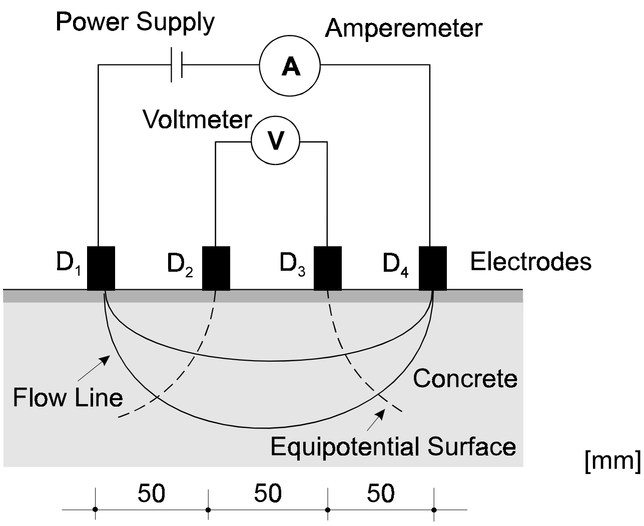
Although concrete resistivity measurement started relatively early, it has not achieved its complete development. Three different ways to measure the concrete resistivity on a given structure were selected:

- Directly on the surface of the structure
- On cores
- Using embedded sensors

Measurements on the surface of the structure

Four-point method

Concrete resistivity can also be measured directly on the surface of the structure by Wenner technique and by the disc method. It was originally developed for geophysical prospecting, but more recently applied to concrete. This method uses four equally spaced point electrodes in contact with the concrete surface.

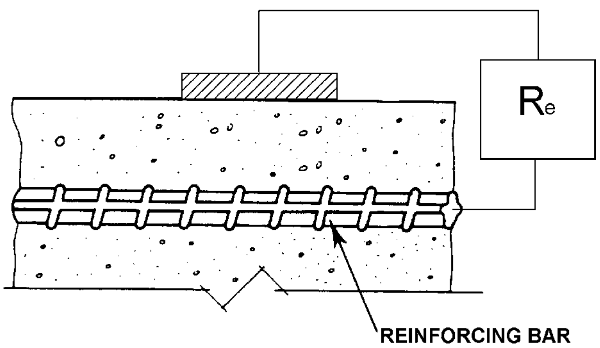


First it is necessary to moisten the electrode tips with a conducting liquid in order to provide a good contact with concrete. A known AC is passed between the outer electrodes and the potential difference is measured between the inner ones. Resistivity is obtained as a function of voltage, current and distance between tips (usually 50 mm). The measurement is carried out with an alternating current with a frequency between 50 and 1000 Hz.

Disc (one point) method

Other option for measuring concrete resistivity is based on Newman's work, and fully developed by Feliú, González and Andrade, which estimates the ohmic drop from the resistance between a small disc placed al the surface of an electrolyte and a much larger counter electrode placed at infinity. If the contribution of the counter electrode resistance to the total resistance is negligible, then it is theoretically demonstrated that electrical resistance is a function of the resistivity of the electrode.

$$\rho = 2 R_e \phi$$



A conductive material disk, a potentiostat and a reference electrode compose the device. After a good contact between electrode and concrete is obtained, a galvanostatic pulse is applied and then, the ohmic drop is recorded from the instant response. As 4-point Wenner method, disk and rebar must no be very close to obtain an accurate measure of bulk resistance. Distance between them has to be at least two times the disk diameter.

Cores

Concrete resistivity can be also measured using cores drilled from a structure. There are different methods developed for laboratory testing as:

- Direct application of Ohm's law
- Deriving mathematical expressions for specific specimens' geometry.
- Through calculation of the 'cell constant' by means of known resistivity electrolytes.

An example of these methods is the Two-Electrode-Method one. The diameter of the cylinders should be 100 mm and their length should be 50 to 100 mm. Two stainless steel plates are pressed to the faces of the core and an alternating current (AC > 1.5 mA, f = 108 Hz) is passed between them.

Embedded sensors measurement

Embedded sensors can be used not to obtain the resistivity but the electrical resistance, by using two or four metal electrodes embedded when it is cast. AC is used with a frequency between 50 and 1000 Hz to measure this parameter. As resistance is function of cell geometry, it must be converted to resistivity according to Wenner theory (4-point method) or based on empirical calibration using liquids of known conductivity in cells of similar geometry (2-point method).

Each sensor consists of several stainless steel rings alternating with insulating plastic rings. They are connected to cables arranged inside the electrode in such a way that they do not influence the surrounding concrete. The gaps of the electrode are filled with epoxy resin. Measurement of AC resistance can be determined between each pair of stainless steel rings by an external AC ohmmeter, and then obtain concrete resistivity by a sensor specific transfer factor. Also moisture content and moisture distribution can be determined indirectly by measuring the depth dependent electrolytic resistance of the concrete.

Efficiency / accuracy

One of the most important problems arising for the measurement of concrete resistivity is its variability with changes in the environment. The factors which influence the resistivity are:

a) Humidity content. The ρ decreases when moisture increases and viceversa.

b) Temperature. The effect of temperature is controversial, as its effect on ρ depends on whether the concrete is shielded or not. That is, whether the water can evaporate or condensate. Thus, an increase of T will decrease the ρ unless it happens the opposite due to the drying induce by T increase. The opposite happens when T decreases, but only until a certain T (around 5ºC), below which the ρ increases so much that the condensation cannot compensate this dramatic increase.

b) Chloride content. The presence of chlorides or any other inorganic compound makes to increase the ρ.

c) Carbonation Conversely, the carbonation aims into an increase in ρ

d) Type of cement Blending agents (fly ashes, slags or silica fume) in general induce an increase of ρ when compared with ordinary portland cement.

e) Porosity The porosity is a consequence of the w/c ratio and to the compaction and curing. An increase in w/c leads into a decrease of ρ.

f) Type of aggregate .The effect of the aggregate type cannot be generically predicted. It will depend on their nature and porosity, but for the same grading, the concrete resistivity is influenced by the aggregate nature.

With respect to the influence of these parameters in the measurement itself, it has to be taken into account, that the accuracy of the measurement is the highest at intermediate values (not too high or low). The exact threshold values will depend on the equipment used, but in general it can be said that those factors (as carbonation) inducing a great increase or decrease (high moisture contents) are those leading to the higher inaccuracy.
Regarding the uncertainties in the measurement itself, the influencing factors are:

- Type of operator and environmental conditions
- Technique
- Instrument

Feliú, Andrade, González and Alonso developed a comparative study of four different measurement methods and for different test conditions. Slabs of 50x50x7 cm were used. Comparative results obtained are shown in next table.

Mean resistivity values (KΩ.cm) obtained through different methods				
Test conditions	Conductivimeter	Wenner's method	Ohm's law application	Disc method
Tap water	13	15.2	13.4	14.5
Concrete, no admixtures, very dry	-	4700	-	6500
Concrete with chlorides, wet.	-	22	-	37
Concrete, no admixtures, wet	-	-	26	29
Concrete, no admixtures, very wet	-	-	8.1	7.2
Concrete with Cl^- very wet	-	-	3.3	3.8

D.6. ULTRASONIC MEASUREMENT

Purpose

The ultrasonic pulse velocity inspection is a non-destructive method which measures the direct compression wave velocity and is used in structural applications to evaluate the condition of materials such as concrete. The method is detailed in ASTM C 597-83 and BS 1881 Part 203.

This method has been used to asses the uniformity and relative quality of concrete and to locate defects (i.e. cracks, voids, etc.) of structural members with two sided access such as slabs, beams and columns. By means of a correct results interpretation, it is possible to obtain information about the existence of corrosion processes into the surveyed structure.

The method exploits the relationship between the quality of concrete and the velocity of an ultrasonic pulse through the material. Many attempts to correlate the pulse velocity with the compressive strength have been done. The basic idea is that pulse velocity is function of material density and stiffness, both of which are related with compressive strength. However, the results obtained in practice have been mixed. There are many variables affecting the concrete compressive strength (water cement ratio, aggregate size and shape, size of sample, cement content, etc.), but not all of them affect the pulse velocity. It is accepted that pulse velocity can be a good indicator of strength gain of concrete at early ages.

In any part of the structure where the concrete appears uniform, the following criteria can be applied to the pulse velocity (TWRL 1980):

Pulse Velocity (m/sec)	Concrete Cover Quality
> 4000	Good
3000 - 4000	Fair
< 3000	Poor

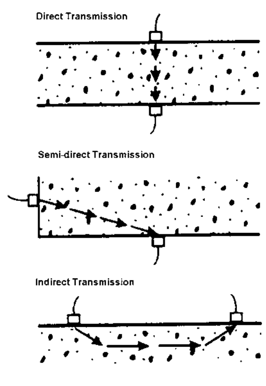
Description

Pulses of compressional waves are generated by a transducer that is held in contact with one surface of the concrete under test. After traversing through the concrete, the pulses are received by another transducer. The travel time *t* is measured electronically. The pulse velocity and arrival time for a stress wave are related according to the equation:

$$V_P = d/t$$

where V_P is the velocity and d is the distance between transducers.

Tests can be performed very quickly. Measurements can be taken by direct transmission, semi-direct transmission or indirect transmission.
The testing apparatus consists of a pulse generator, a pair of transducers (transmitter and receiver), an amplifier, a time measuring circuit, a time display unit, and a connecting cables. A very low frequency (range of 10 to 150 kHz), high energy pulse is required because of the high attenuative nature of concrete.



Efficiency / accuracy

The accuracy of the measurement is dependent upon the ability of the equipment and the user to determine precisely the time of arrival of the wave at the receiver and the distance between transducers. The amplitude of the received signal depends on the travel path length and by the presence and degree of flaws in the concrete tested.

The achieved repeatability of test results for path lengths from 0.3 to 6 m through sound concrete is within 2 %. In cases of test through concrete with flaws, the variation of the results may be as large as 20 %.

D.7. IMPACT-ECHO

Purpose

Impact-Echo is a non-destructive technique based on the use of transient stress (sound) waves. A short duration mechanical impact at a point of the surface is used to generate stress waves. These waves propagate through the structure and are reflected back to the surface by internal flaws or interfaces and by external surfaces of the structure. One of the advantages of the Impact-Echo method over the Ultrasonic Pulse Velocity method is that only one side of the structure needs to be accessible for testing.

Main objective of the Impact-Echo method is the assessment of concrete structures. The different applications can be classified as follows:

1. Concrete thickness measurement;
2. Mapping of flaws (voids, honeycomb, delaminations, cracks, etc.); and
3. Acoustic behaviour of interfaces between materials, such as in stratified structures, repaired structures or in reinforced structures.

A very common problem in construction activity is how determine the concrete thickness of structures that only have one accessible side. Impact-Echo method generates low frequency stress waves that can penetrate within concrete up to approximately 2 meters. With this technique it is possible determine the construction thickness in a wide range of structure typology as concrete pavements, retaining walls, shaft liners, etc.

Concrete structures are exposed to degradation processes that can arise in a loss of section thickness. The extent of deterioration of the back face of structural members due to chemical attack or freezing and thawing can be assessed using the Impact-Echo method.

When embedded reinforcement steel is corroding, expansive forces are generated due to the presence of corrosion products. It leads to bond deterioration and a delamination process. Where the concrete surface is exposed, shallow delamination can be easily detected by visual inspection or chain drag methods. However, where the delamination is deep or there is an overlay over the concrete surface, only advanced methods as Impact-Echo are able to detect it.

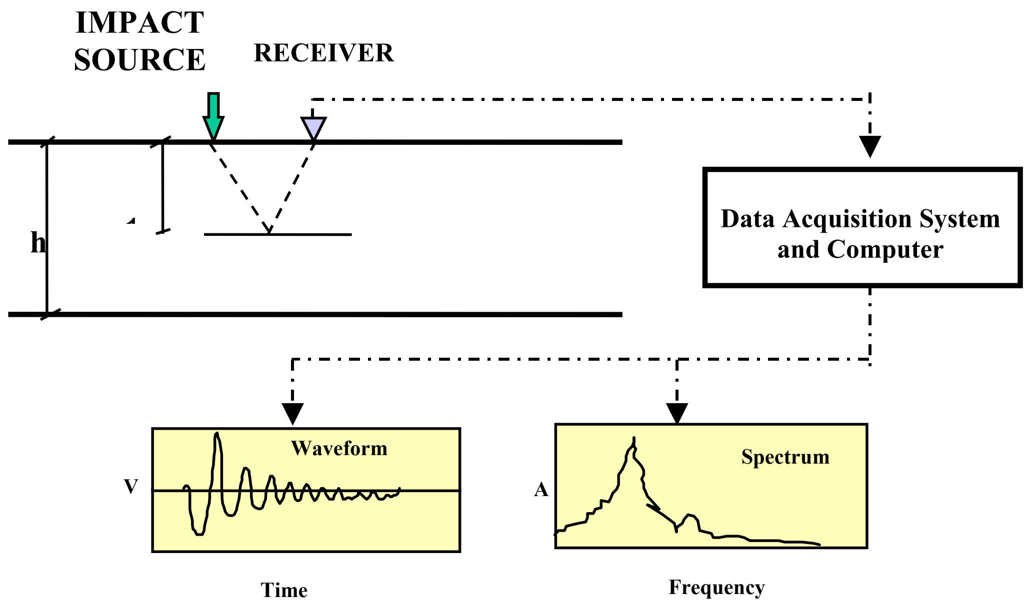
Impact-Echo method can be used to locate hidden damage like: the tracing of the profile of a crack, detection of internal defects of manufacture, crack depth measurement or presence of honeycombing.

This technique enables to detect the presence of voids below slabs-on-grade, behind retaining walls and around tunnels systems, including concrete shafts and drainage systems. It has also been used to verify complete filling of the voids before, during and after repair. In some cases Impact-Echo

technology can detect voids in grouted ducts. Where the duct is not filled, the impact waves are distorted.
Impact-echo signals can be used to detect the quality of bond between different layers. This technique has been used on tunnel liners to verify the quality of the bonding.

Description

Transient stress waves are introduced into a structure by mechanical impact at a point on the surface. These waves travel into the structure and are reflected by internal defects or external boundaries, propagate back to the surface and again into the test structure by which a transient resonance condition is set up as multiple reflections occur. A displacement transducer located closed to the impact point records the surface displacements caused by the arrival of the reflected waves.



$$f = \frac{V_p}{2T}$$

Schematic illustration of an Impact-Echo test.

The frequency of wave arrivals at the transducer is determined by transforming the record time domain signal into the frequency domain using the fast Fourier transform technique. The frequencies associated with the peaks in the resulting amplitude spectrum represent the dominant frequencies in the waveform. Figure 1 shows an illustration of an impact-echo test on a solid plate with a flaw, the waveform obtained and the corresponding amplitude spectrum.

An impact-echo test system has three components:

- An impact source (small steel balls, instrumented hammer, etc.);
- A displacement transducer;
- A data acquisition system and a computer with waveform analyser.

Efficiency / accuracy

Impact-Echo method generate low frequency stress waves (< 80 kHz) that can penetrate within concrete up to approximately 2 meters. It has successfully used to determine the thickness of many concrete structures with errors less than 5 %.

The Impact-Echo method has many applications locating flaws or damage in concrete, but it also important to realise that often the method is used most effectively in connection with other non-destructive techniques in an evaluation of a concrete structure. Based on core sampling or cutting, the impact-echo measurement results can be quantified to evaluate the practical effects of detected damage.

In practice, the method is not accurate all the times. Some uncertainty is always connected with the use of the method, both in the limitations of the method and in the limitations of the user's experience. The interpretation of the test results is the most important problem.

Impact-Echo is a very sensitive method, even reacting to micro-defects. The size of the target detected depends of the wavelength which depends of the impact contact time. As the contact time decreases, smaller defects can be detected. However, as contact time decreases the penetrating ability of the stress waves also decrease.

ANNEX E. STRUCTURAL SAFETY

E.1. Introduction

The main purpose of a structural assessment is to ensure a determinate level of safety in a existing structure. About how to perform such inspection, this manual presents several steps and guides that offers the inspector a complete procedure for assessing concrete structures affected by rebar corrosion.

However, once the material characteristics have been obtained and the section nominal efforts too, both terms have to been compared in order to assess the current safety level of the structure.

E.2. The limit state philosophy

The common design procedure in Europe is the limit states. The limit states separate a favourable state from another unfavourable or undesirable. Several types of limit states has been collected though time although the more extensive classification establishes tow main types of limit states: Serviceability and Ultimate. Table E1 shows the typical limit states for structures:

Table E.1 *Typical limit states in structures*

Limit State type	*Description*	*Examples*
Ultimate	Collapse of all or part of a structure	Rupture, collapse, plastic mechanism, fatigue, fire, etc.
Serviceability	Disruption of normal use	Excessive deflections, vibrations, local damage, etc.

In order to assess if a current structure overpasses the analysed limit state equation (1) has to be checked. In (1) ***S*** are the load effect on the structure (shear, bending moment, torsion, load period, etc.) and ***R*** are the load effect resistance of the structure (ultimate shear, ultimate bending moment, ultimate torsion effort, eigen period of the structure, etc.).

$$R \geq S \qquad (1)$$

However, it is desirable to maintain both terms of equation (1) separated enough to ensure a lower risk of reaching the analysed limit state. For achieving this purpose the values used in equation (1) must be *design values* and not nominal values (although both could be coincident in some cases), equation (1) is transformed into (2) for safety assess. Several levels of safety can be used in the assessing procedure (Table E2).

$$R_d \geq S_d \qquad (2)$$

Table E.2. *Level of safety measurement*

LEVEL	*Description*	*Data employed*	*Mechanism*
I	Normal codes prescription	Default data	Partial safety factors
II	Second Moment theory	Normal Statistical distribution of data	*Nominal* failure probability P_{Fn}.
III	Simulation or transformation (FORM – Monte Carlo)	Any statistical distribution of data	Failure probability P_F

IV	Any of the above, plus economic data	Minimum cost or maximum benefit

a) Level I methodology

The most common technique is the use of partial safety factors, and is the default technique in structural design. In this case equation (1) has to be verified using the partial safety factors, equation (2).

$$R\left(\frac{f_K}{\gamma_{m1},\gamma_{m2},\gamma_{m3}}\right) \geq S\left(\gamma_{f1},\gamma_{f2},\gamma_{f3},Q_K\right) \qquad (2)$$

Where ***R()*** and ***S()*** are the resistance and load effect functions which convert the strength or action into resistance and load effect respectively, and γ_m and γ_f are the partial safety factors of materials and actions respectively.

When more than a single action is acting on the structure, the *combination coefficients* Ψ are used. Thus, equation (2) is transformed into (3).

$$R\left(\frac{f_K}{\gamma_{m1},\gamma_{m2},\gamma_{m3}}\right) \geq S\left(\sum_{i>1}\gamma_i \Psi_i Q_K\right) \qquad (3)$$

Table E.3 shows the default values of EC1 for new structures.

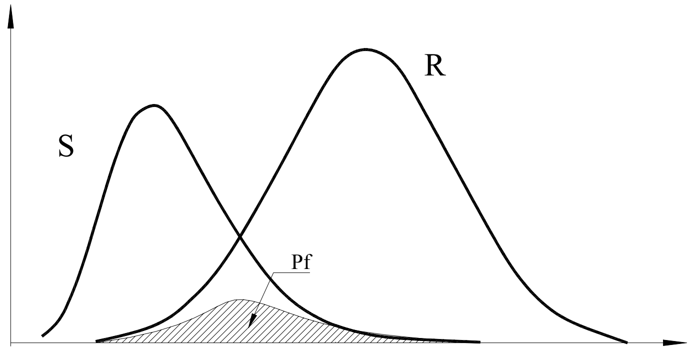
Limit State	*Load*	**Favourable**	*Unfavourable*
Ultimate	Permanent	$\gamma = 1,00$	$\gamma = 1,35$
	Variable	$\gamma = 0,00$	$\gamma = 1,50$
	Accidental	$\gamma = 0,00$	$\gamma = 1,00$
Serviceability	Permanent	$\gamma = 1,00$	$\gamma = 1,00$
	Variable	$\gamma = 1,00$	$\gamma = 1,00$
	Accidental	$\gamma = 1,00$	$\gamma = 1,00$

It is commonly accepted, that these values can be changed (reduced or increased) when assessing an existing structure. However there are not a simple way of reducing these values and a correlation with level II methods must be done in order to ensure a relative equal safety level for both structures (a new one and an existing one).

b) Level II methodology

Level II is based on a statistical quantification of the limit state analysed. Thus, the failure probability may be obtained. Each variable in the limit state formula must be statistically characterised (at least with their mean value and their standard deviation). Figure E1shows the statistical quantification of resistance and action effects on a determinate section and the join failure probability. In order to compare the actual safety level in the structure (measured in terms of failure probability), background documents of current codes provides the nominal failure probability at which have been calibrated. As an example Eurocode 0 is said to be calibrated at a nominal failure probability of $7{,}2\ 10^{-5}$.

When using the level II methodology, better than calculation the failure provability, the *reliability index* (β) is calculated. The *reliability index,* is univocally joined with the failure probability by means the cumulative Gauss distribution Φ. Thus, the value of P_F may be calculated with the aid of equation (4) and a tabulated cumulative Gauss distribution.



$$P_F = \Phi(-\beta) \tag{4}$$

The *design value* of each variable is the more probably combination of values that makes the structure to reach the limit state. For each variable may be obtained (always supposing normal distributions for ***R*** and ***S***) by means equation (5), where

$$\begin{aligned} R^* &= \mu_R - \alpha_R \beta\, S_R \\ S^* &= \mu_S - \alpha_S \beta\, S_S \end{aligned} \tag{5}$$

Where:

μ_R is the mean value of the resistance.

μ_S is the mean value of the load effect

S_R is the standard deviation of the resistance

S_S is the standard deviation of the load effect

$\alpha_R = \dfrac{S_R}{\sqrt{S_R^2 + S_S^2}}$ the importance factor for resistance, and

$\alpha_S = \dfrac{S_S}{\sqrt{S_R^2 + S_S^2}}$ the importance factor for action effect.

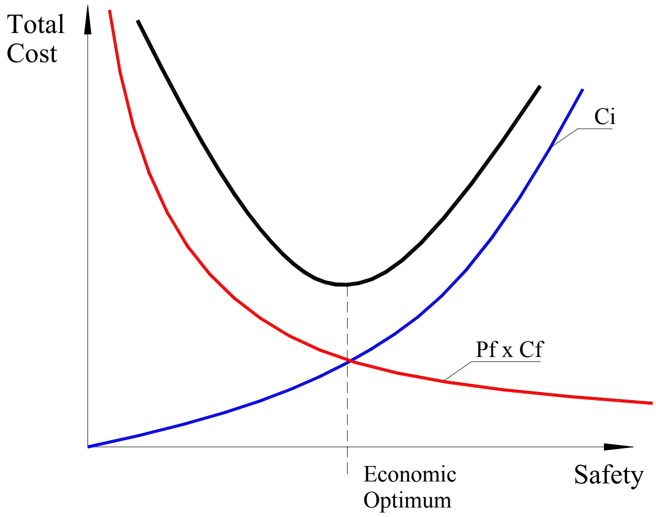
(It can be easily seen that $\alpha_R^2 + \alpha_S^2 = 1$)

c) Level III methodology

The level III methodology is an extension of level II, using for each variable their statistical distributions, that usually is not a normal distribution. For this methodology, two types of approach are commonly used the Monte Carlo procedure and the transformation one.

Monte Carlo procedure is an artificial simulation, using the variables involved in the problem. The main handicap of this procedure is the time consuming. Transformation algorithms allow using equivalent normal distributions taking into account the original distribution. Such procedures can be the FORM or SORM algorithms, for these procedures equivalent equation (5) may be obtained for each variable and therefore, the *design value* of each variable may be achieved. Both methods here presented (Monte Carlo and transformation methods (FORM/SORM)) will produce as direct result the failure probability of the element is achieved and therefore a direct comparison with nominal values obtained.

d) Level IV methodology



The level IV includes an economical optimisation of the structure too. The total cost of the structure can be obtained with the aid of equation (6), where C_i are the initial cost of the structure, C_m are the maintenance costs, P_F is the failure probability and C_D are the economical cost of failure. The product of P_F by C_D is known as the *risk*, and is a continuous decreasing function due to P_F decreasing. On the other hand C_i is a continuous increasing function, because a safer structure is more expensive. Therefore the sum of both terms including a relative constant value of maintenance will have a minimum. This point should be the key point for designing and assessing structures (figure E2).

$$C_T = C_i + C_m + P_F\ C_D \qquad (6)$$

E.3. The partial safety factors in ENV 1992:1

The first step into reliability analysis is the knowledge of the source (or an attempt to justify) the actual partial safety factors in ENV 1992:1. Two steps must be done, first of them is the calculation of psf for resistance γ_R and the corresponding to actions γ_S.

a) Derivation of γ_R

Two main values are going to be derivated the first of them is corresponding to the steel yield strength γ_S and the second one the corresponding to the concrete strength γ_C.

The value for γ_S may be calculated through an analysis of the reliability study of a beam under flexural failure.

The *default* value for random variables on the resistance side can be assumed to be:

Variable	*Description*	*Distribution*	*CoV (%)*
F_Y	Steel yield strength	Log – normal	5 – 10
F_C	Concrete strength	Log – normal	30 – 10
H	Geometry	Normal	5
ξ	Model error for flexural failure	Normal	5

Once, the calculation is performed classical solutions for the resistance variables are about:

- $\alpha_{Fy} = 0{,}8$.
- $\alpha_H = 0{,}4$.
- $\alpha_{Fc} = 0{,}2$.
- $\alpha_\xi = 0{,}2$.

Neglecting the alpha factor for concrete strength, and merging the alpha factor for geometric (α_H), model (α_ξ) and steel yield strength into one value by merging their CoV values with the square root role one can obtain (7)

$$CoV_{Fy} = \sqrt{0{,}08^2 + 0{,}05^2 + 0{,}05^2} = 0{,}106 \approx 0{,}10 \tag{7}$$

Then applying equation (5) with a minimum reliability index of $\beta = 3{,}8$ one can yield into (8):

$$F_{yd} \approx \mu_{Fy} e^{-\alpha_{Fy}\beta\, CoV_{Fy}} \tag{8}$$

Expression (8) for classical steel reinforcements of $F_{yk} = 500$ MPa provides a value of 420,6 MPa, the partial safety factor can be now calculated as (9).

$$\gamma_S = \frac{F_{yd}}{F_{yk}} = 1{,}18 \tag{9}$$

Which is vary close to the normal value of $\gamma_S = 1{,}15$. The overestimation of the value is due to the merging value of (α_H) and (α_ξ) for the sake of simplicity but the result is certainly good.

Regarding the concrete strength, the partial safety factor for concrete strength is classically divided into two terms, γ_{C1} and γ_{C2} (10). The first term tries to represent the relation between the design value of the concrete strength and the value of the characteristic strength in the structure, while the second factor is the relationship between the concrete strength in the structure and the concrete strength in the specimens used for quality control (cylindrical or cubic).

$$\gamma_C = \gamma_{C1}\,\gamma_{C2} \tag{10}$$

The value of γ_{C1} can be calculated easily by a reliability analysis of a reinforced column under bending moment and axial force, in this case the literature review suggests to use a CoV for the model uncertainty of 10%. The results for the alpha factors are complementary of the flexural mode and about:

- $\alpha_{Fy} = 0{,}2$.
- $\alpha_H = 0{,}2$.
- $\alpha_{Fc} = 0{,}8$.
- $\alpha_\xi = 0{,}2$.

Merging again the values of the model uncertainty and geometry with the concrete dispersion yields to (11).

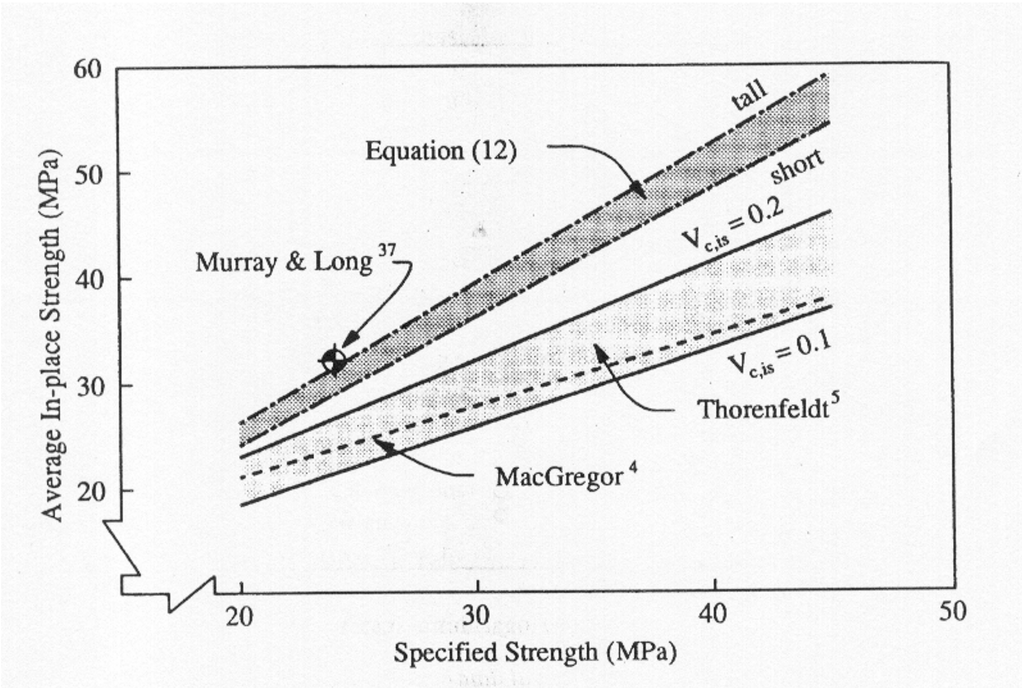
$$CoV_{Fc} = \sqrt{0{,}15^2 + 0{,}10^2 + 0{,}05^2} = 0{,}18 \tag{11}$$

Now, applying equation (5) with a minimum reliability index of 3,8 and α_{Fc} = 0,8 one can obtain for a concrete strength of 25MPa a design value of 18,11 MPa. Thus, the value of the first term in (10) can now calculated as (12).

$$\gamma_{C1} = \frac{F_{Cd}}{F_{C,Sturcture}} = 1{,}38 \tag{12}$$

For the calculation of γ_{C2} several tests have been performed in Germany with cubic specimens and in Canada with cylindrical ones by Mirza y McGregor. For all curves obtained the inferior quantile that transforms the specimens strength into *in situ* strengths results for 28 days equal to 0,85. Thus the final value of the partial safety factor for the concrete strength results (13).

Relationship between in situ strength and specimens strength (several authors)



$$\gamma_C = \gamma_{C1}\,\gamma_{C2} = 1{,}38\,\frac{1}{0{,}85} = 1{,}62 \tag{13}$$

Again, the value obtained is very close to the prescribed value of ENV 1992:1. In this case the overestimation of the variation coefficient for the importance factor of 0,8 is greater than before.

b) Derivation of γ_S

On the action side, the same principles applies. One may take from ISO 2394 following values for the alpha values. These values are fully concordant with previous used for derivation of γ_C and γ_s .

	Load	Resistance
Dominant variable	α_i = 0,70	α_i = 0,80
All other variables	α_i = 0,28	α_i = 0,32

For each type of structure the loading pattern should be found depending on the type of use and load history on it. An easy explanation of the source of each partial safety factor is not available although it could be done depending on the structure type, several background documents have been produced by the JCSS regarding this topic.

- Dead load

Justification for dead load could be found on the combination with environmental actions (snow, wind). Thus, regarding table above two types of combinations should be done.

- Dead load to be *dominant*

$$\gamma_G = \frac{1 + 0{,}7 \cdot 3{,}8 \cdot 0{,}15}{1} = 1{,}39 \tag{14}$$

$$\gamma_W = \frac{1 + 0{,}28 \cdot 3{,}8 \cdot 0{,}4}{1} = 1{,}42 \tag{15}$$

- Wind load to be *dominant*

$$\gamma_G = \frac{1 + 0{,}28 \cdot 3{,}8 \cdot 0{,}15}{1} = 1{,}15 \tag{16}$$

$$\gamma_W = \frac{1 + 0{,}7 \cdot 3{,}8 \cdot 0{,}3}{1} = 1{,}8 \tag{17}$$

The value for the dead load γ_G = 1,35 is close to the value obtained in this example.

- Variable loads

A Gumbel distribution for the time varying load is reasonable (19).

$$F_Q(x) = e^{-e^{-\alpha(X-u)}} \tag{19}$$

Where α and u are the Gumbel parameters, related to the mean and standard deviation of the 50 years loads return period. Expression (19) can be transformed into (20) where k_σ represents how many standard deviations are separated from the mean the point in evaluation X.

$$F_Q(x) = \exp\left[-\exp\left(-0.577 - \left(\frac{\pi}{\sqrt{6}}\right)k_\sigma\right)\right] \tag{20}$$

For the calculation of design value, equation (5) applies and with a α_Q = -0.7 and β = 3,8 we have (21) by rearrangement.

$$Q_D = \mu_Q + 3{,}86\sigma_Q \tag{21}$$

In the case of Gumbel distribution the characteristic value (fractil of 5%) results (22).

$$Q_D = \mu_Q + 1{,}86\sigma_Q \tag{22}$$

Finally the safety factor for imposed loads can be derived as (23), where for a bias of 1,05 and a CoV_Q of 30% a value of 1,5 is obtained.

$$\gamma_Q = \frac{1+3{,}86CoV_Q}{1+1.86CoV_Q} \tag{23}$$

E4. The safety of existing structures

The most important difference between assessing and designing is the information level about the structure. Table E.4 shows the main differences between the information of each point at the design and at the assess level.

Table E4. *Main differences between designing and assessing*

	Design	Assess
Action effects	Assumed Code based	Assumed (code based) Owner's requirement Known (measured))
Element resistance	Defined as project requirements	Known trough *in situ* test

The common procedure for structural designing use to be the employ of standards and practice code for the action effect evaluation, in function of the final structure's use. In those cases where no code is available an assumption should be made (generally on the safety side). When assessing structures, the action effects, such as self – weight load, will be determined directly from measurements of geometrical dimensions on elements. For load such as dead load, the same procedure can be made taking into account the actual state of the structure. Finally, the live load can be assumed to be the same as proposed by the design codes, however there are some practice codes of assessing where reduced loads are proposed for assessing. As a last resource the load level may be directly accorded with the structure owner.

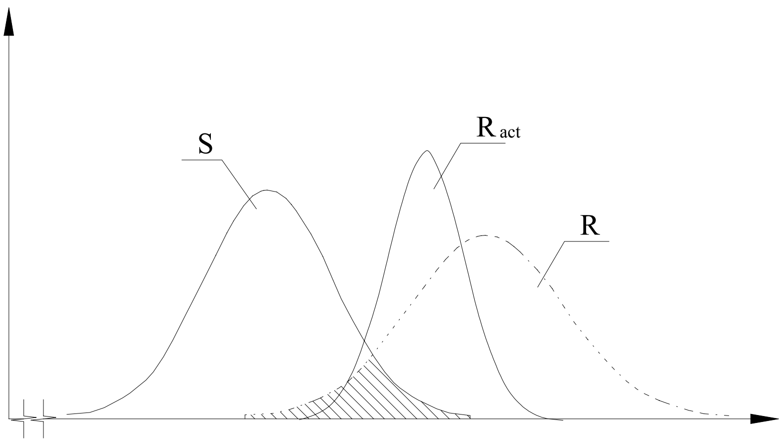
On the other hand, the resistance of the element that form the structure, are prescribed as requirements in the design phase. However, in the assessing phase, the testing over the structure, allows to know the material properties of the studied element.

Influence of information in the safety level

Once, it can be accepted that the information in the assessing phase may be higher than in the designing one, let see what is the influence of this information in the safety level of the structure.

Typical partial safety coefficients that are present in codes, are based on a high scarce that allows many structures to be designed by those codes. Thus, the distribution of ***R*** and ***S*** should be wider enough to cover a great number of cases. When information is actualised the scarce of both terms ***R*** and ***S*** can be reduced (the most common technique is reducing ***R*** scarce due to the high cost of reducing ***S***) and the failure probability will change, depending of the relation between ***R*** and ***S***. Figure E3.

Thus, the variable actualisation will produce an actualisation in the failure probability and therefore in the structural safety.



E5 Modifying the partial safety factors

Once, a critical value of failure probability has been established, using default values in codes or using a *self calibration* procedure, the partial safety factors can be modified taking into account the information available in the structure.

As was explained before equation (2) must be checked with *design values*. This *design values* are obtained with the classical partial safety factors (level I) as:

$$R_d = \frac{R_{nom}}{\gamma_R} \qquad\qquad S_d = \gamma_S S_{nom} \tag{7}$$

Where R_{nom} and S_{nom} are nominal values of the resistance and the action effect (usually these nominal values are the characteristic values).

When using level II or III methodology equation (2) is verified using the *designing values* too, which are in this case (8).

$$\begin{aligned} R^* &= \mu_R - \alpha_R \beta\ S_R \\ S^* &= \mu_S - \alpha_S \beta\ S_S \end{aligned} \tag{8}$$

Where β can be the minimum allowed by code for every limit state.

Thus, a direct correlation can be established between equation (7) and (8), because the *designing values* are similar in both cases.

$$R_d = \frac{R_{nom}}{\gamma_R} = \mu_R - \alpha_R \beta\ S_R \qquad\qquad \gamma_R = \frac{R_{nom}}{\mu_R - \alpha_R \beta\ S_R} \tag{9}$$

$$S_d = \gamma_S S_{nom} = \mu_S - \alpha_S \beta\ S_S \qquad\qquad \gamma_S = \frac{\mu_S - \alpha_S \beta\ S_S}{S_{nom}} \tag{10}$$

And, if the nominal values are considered to be the characteristic values eq. (9) and (10) can be written as (11) (12).

$$\gamma_R = \frac{\mu_R - 1{,}6449\ S_R}{\mu_R - \alpha_R \beta\ S_R} \tag{11}$$

$$\gamma_S = \frac{\mu_S - \alpha_S \beta\ S_S}{\mu_S + 1{,}6449\ S_S} \tag{12}$$

Practical application

Although chapter above presents a rational process of developing the new partial safety factors, at the moment, this procedures has been only applied in calibration process for recommendations and it is not a common working procedure. Thus, several recommendations are available in the bibliography and may be used for direct assessing. However after an extensive investigation of the recommendation around the world this points must be remarked:

1. **The code's behind philosophy.**

   Every evaluation recommendation (for building or bridges) is based in the code that rules the design phase. Thus, classical formulas for bending and compression are physical based, but others based in empirical evaluations (such as shear, punching, bond, fatigue, etc.). The direct application of the reduction coefficients is difficult if the code is different.

2. **The safety format**

   Although many of the codes are based on the CEB safety format (similar to equation (2)), there are some exceptions that use another safety format. Some examples could be the National Building Code of Canada, the Load and Resistance Factor Design (LRFD) of US code or the DIN1045 for concrete design in Germany.

3. **The load level**

   As was exposed before, code should be calibrated to a certain failure probability and therefore, both the resistance and the load effect must be coherent. Thus, although it could be tempting to use every date available, a certain care must be taken mixing both terms of basic equation (1).

Finally, next table presents an extensive summary of the available bibliography where educed factors can be obtained or reduced load evaluated.

Title	*Editor*	*Country*	
Background documentation Eurocode 1 (ENV 1991:1)	JCSS		1996
Probabilistic Model – Code	JCSS		
Assessing recommendations for bridges	Ministry of transport	Germany	1993
Recommendations for load bearing capacity determination in steel structures for railways.	UIC	---	1986
Asses of concrete structures in buildings	ACI	US	1999
BD & BA Documents, Design manual for road and bridges. Volume 3 Highway structures: Inspection and maintenance.	Highways Agency	UK	1992

E6 Bibliography

Melchers, R.E. *Structural Reliability: analysis and prediction.* Wiley, New York. 1983.

Background Document of ENV 1991. *Project Team of EC 1.*

ANNEX F. STRUCTURAL ASSESSMENT

This annex has been structured in four main parts. The former describes the experimental work and the theory in which it is based on. Point F.2 if focused on the effects of corrosion on materials (steel and concrete) and points F.3 and F.4 lead with the application of the U.L.S. and S.L.S. Theory

F.1 EFFECTS OF CORROSION ON STRUCTURES

F.1.1 Reinforcing Bars

Depending on the characteristics of the aggressive agent, corrosion of reinforcement steel and its influence on the cross section becomes very different. While homogeneous attack penetration occurs in carbonated concrete, chlorides usually produce localised attack known as pitting, which causes a significant section decrease. Once the depth of the attack penetration has been obtained, the residual bar diameter can be estimated by means of the following expression:

$$\phi_t = \phi_0 - \alpha P_x$$

where α is a coefficient that depends on the type of attack. When homogeneous corrosion occurs, α is equal to 2. However, when localised corrosion occurs, α may reach values up to 10. A conservative value of the residual section at pits can be also predicted by the above expression.

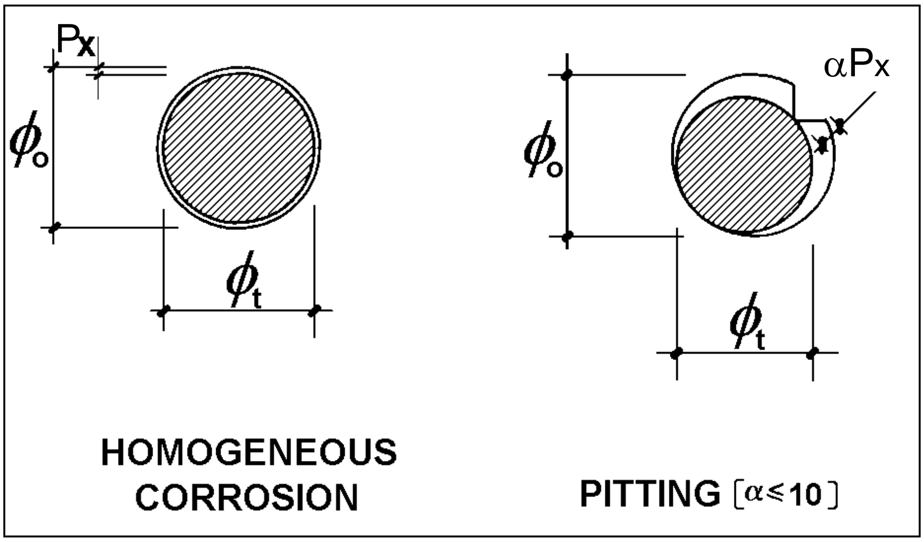


Figure F1 *Residual reinforcing bar section.*

As an example, figure f.2 shows the decrease in cross sectional area of bars with diameters of 6 and 20 mm and for a representative corrosion rate value $I_{corr}^{rep} = 1\ \mu A/cm^2$. Two corrosion conditions are represented in this figure: homogeneous corrosion and maximum pitting (α =10). Whereas homogeneous corrosion is negligible in terms of section reduction for high diameter bars, pitting corrosion in small diameters has a relevant effect.

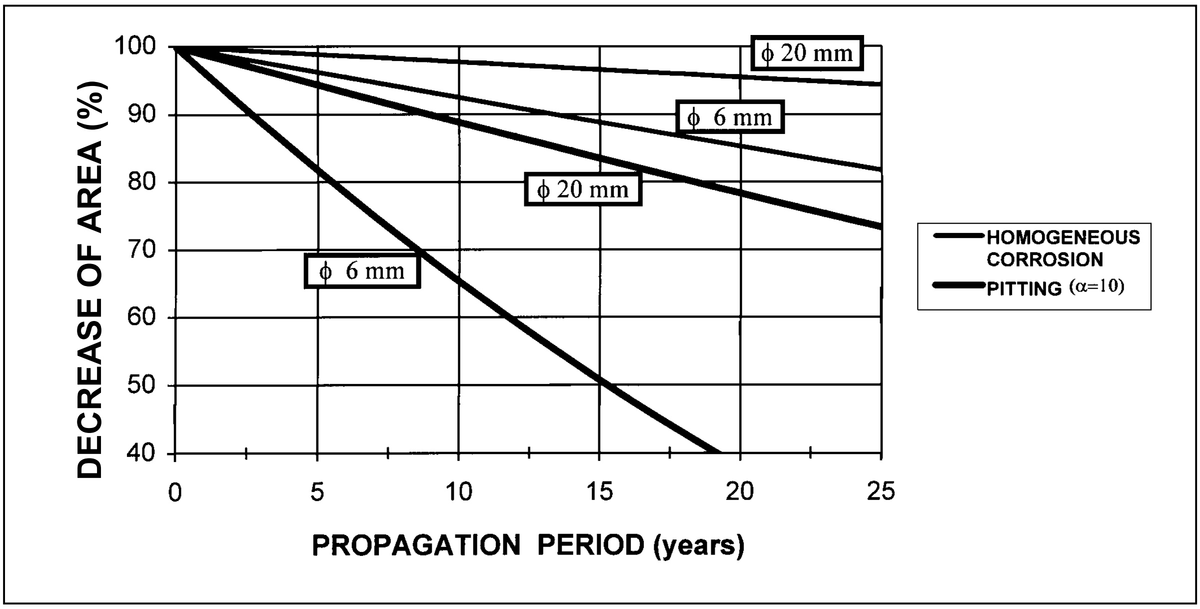


Figure F2 *Decrease of bar sectional area for* $I_{corr} = 1\ \mu A/cm^2$.

F.1.2. Mechanical properties

Corrosion also affects the mechanical properties of the steel. A significant reduction of the elongation at maximum load was found in accelerated tests[f.2] which reached reductions in the elongation of 30 and 50% for cross section losses of 15 and 28 %.

The values of elongation at maximum load remained in general above the minimum value indicated in the Eurocode 2 for high ductility steel [f.3] in which the characteristic elongation value at maximum load higher than 5% is requested.

However, this reduction may affect the capacity of the bending moment redistribution in corroded structures. Thus, the values of maximum ratio δ of the redistributed moment to the moment before redistribution included in Eurocode 2 should be taken with caution.

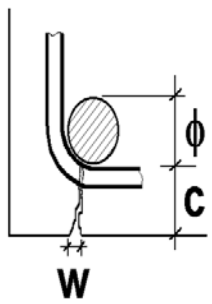
It has been also observed a minor tendency to the reduction of the yield stress and tensile strength of the corroded steel. However, these results should be also taken with reserve because:

- It's difficult to estimate the true residual bar cross section at the failure section.
- Some authors[f.4,f.5] used the initial cross section area of the rebar to calculate the yield stress and the tensile strength instead of the reduced cross section [f.2,f.6]. By this way, the effect of the steel strength deteriorarion is not separated from the influence of cross section loss and most of the strength decreasing migth be due to the reduction of the cross section.

F.1.3 Cracking of concrete cover

The oxide generated during the active corrosion of the reinforcing bars induces a pressure on the surrounding concrete which in most cases leads to the cover cracking (Figure f.3). These cracks cause the loss of concrete integrity which reduces the concrete contribution to the load bearing capacity and affects the external appearance of the structure.

An experimental work was carried out on accelerated and non-accelerated corrosion tests to relate the initiation and evolution of the concrete cracking to the level of corrosion[f.7,f.10]. Time needed for the initiation of the crack mainly depends on the concrete cover/bar diameter ratio and on the quality of the concrete. Porosity was found the more relevant concrete parameter. However, the splitting tensile strength was considered in the project as the appropriate variable to reflect the quality of the concrete because both porosity and tensile strength are related and more data on splitting tensile strength was collected during the experimental work.



Besides, the crack width evolution mainly depends on the position of the bar in the concrete element (top or bottom cast position), and on the corrosion rate I_{corr}, although this influence was negligible for usual corrosion rate values ranging between 0.1-0.2 and 1-2 $\mu A/cm^2$.

Figure F3 *Cover cracking.*

As an example, figure f.4 shows the initiation and the evolution of crack width for three values of the splitting tensile strength and c/ϕ ratio. Once the aggressive reaches the bar, cracks appear very soon in high quality concrete with low c/ϕ values. Conversely, about 0.06 mm are needed in low quality concrete with high c/ϕ values, because the corrosion products can easily diffuse through the concrete cover due to the high porosity in low quality concrete.

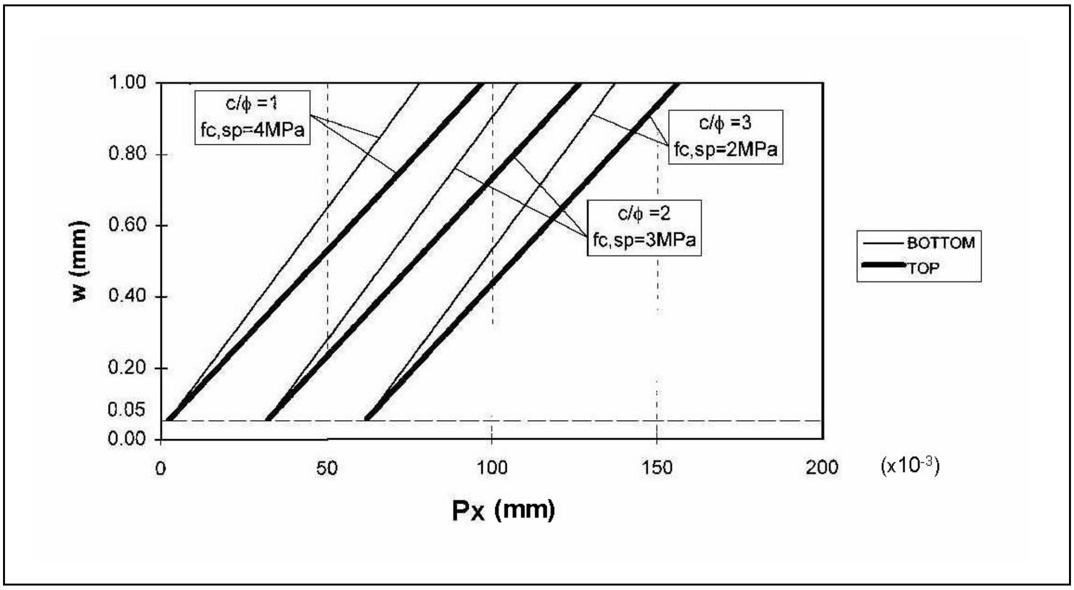


Figure F4 *Crack width versus attack penetration (bar radius decrease).*

During the research work carried out on corrosion and cracking, the spalling of the concrete cover was not reached although significative crack widh was produced in some specimens. These specimens were submited to corrosion when they were unloaded. However, the interaction between the application of the loads on the structure and the corrosion of the reinforcement can lead to cover spalling, as it was verified in beams and columns with corroded elements

F.1.4 Bond

Bond between reinforcement and concrete causes the bar end anchorage and the composite interaction between steel and concrete at intermediate parts of the bar. However, corrosion reduces bond due to the weakening of the bar confinement produced by both the concrete cracking and the stirrup corrosion. Consequently, anchorage failure and loss of composite interaction may result.

A research work was carried out to establish the relationship between corrosion and bond deterioration[f.11,f.14]. The experimental work was mainly based on tests carried out with cubic specimens reinforced with four bars in their corners, with and without stirrups, thus reproducing a beam submitted to constant shear force (Figure f.5). These tests allowed realistic bond strength values to be developed which could be applied to the design and assessment of concrete structures. Neither the concrete quality nor the cover size/bar diameter ratio influenced the residual bond strength if the cover is cracked by reinforcement corrosion.

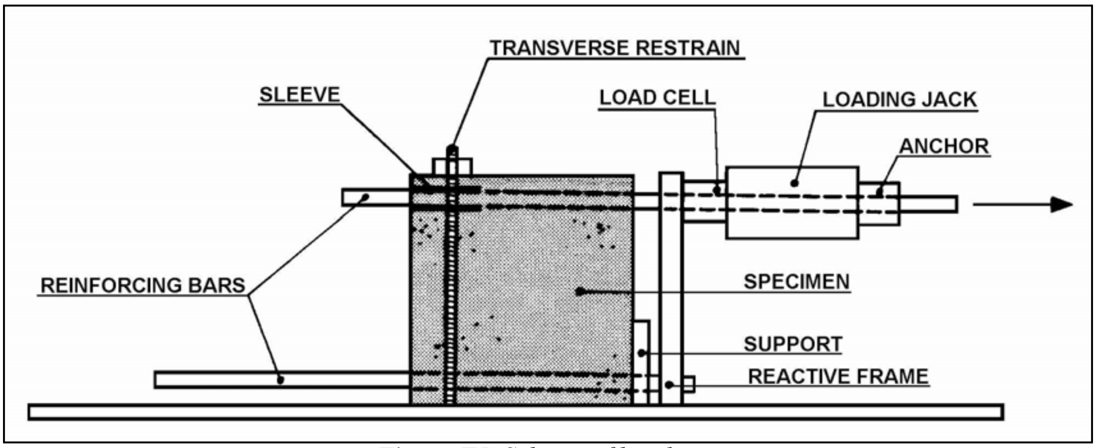
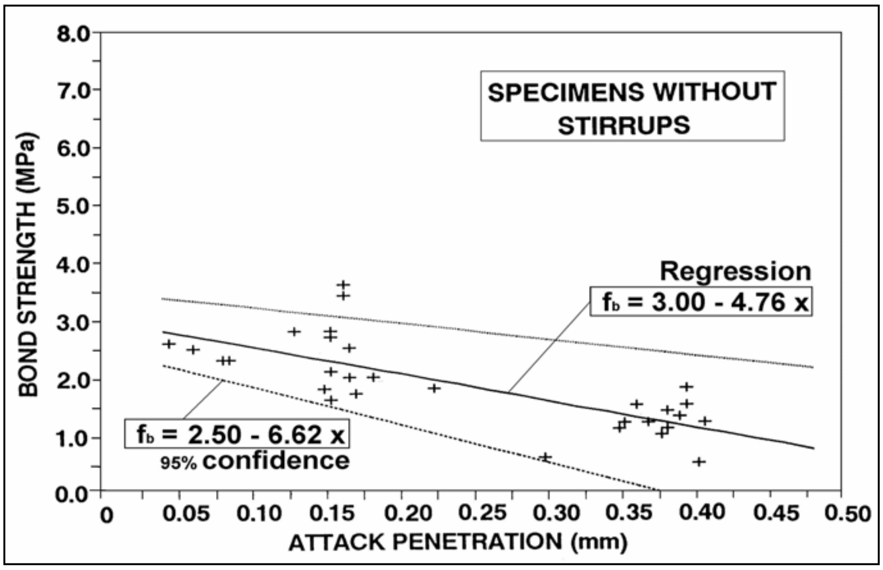


Figure F5 *Scheme of bond test.*

Figure f.6 shows the bond strength results versus the attack penetration (bar radius decrease) corresponding to corroded bars in two types of specimens with and without stirrups. The amount of stirrups varied in the first type of specimens but they were always higher that the minimum ratio ($\rho>0.25$) specified in Eurocode 2 for the anchorage length. Only the main bars were corroded and the stirrups remained practically non corroded. Figure f.6 includes the straight lines obtained by a linear regression analysis, the lines corresponding to 95% confidence interval and their expressions.



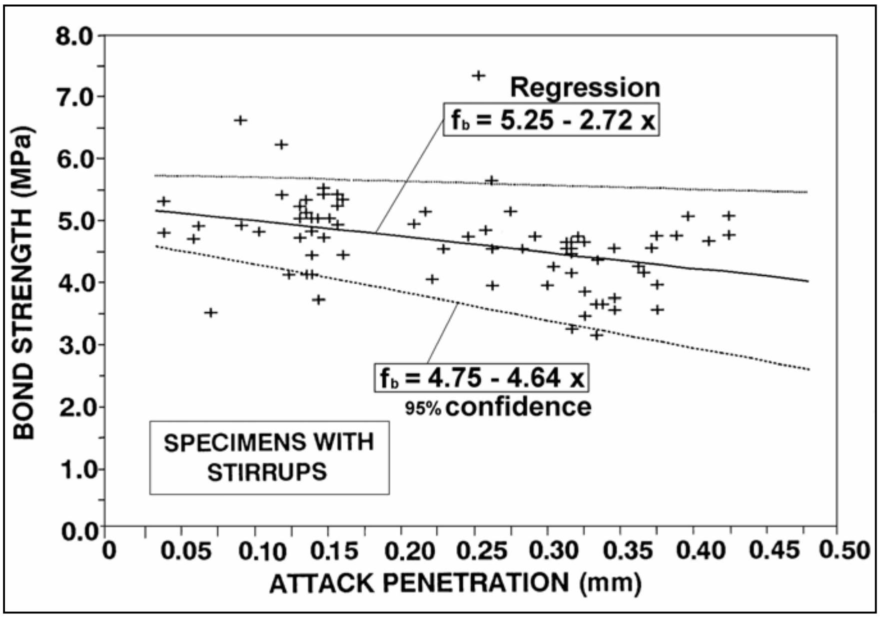


Figure F6 *Bond strength versus attack penetration (decrease of bar radius).*

The expressions for the lower bound of the 95% confidence interval are recommended for assessment of deteriorated concrete structures, either with a minimum amount of stirrups higher than that specified in Eurocode 2 (bottom figure) or without stirrups (top figure). Although the experimental values of the attack penetration P_x ranged between 0.04 and 0.5 mm, it seems appropriate to extrapolate their results till P_x = 1.0 mm.

If the initial stirrups section at anchorage length was lower that the minimum or, if being initially higher, corrosion reduced it to a value lower than the minimum, then the residual bond strength would correspond to intermediate values between those obtained with and without stirrups (Figure f.6).A proposal was developed to cover these intermediate values, considering as boundary values those obtained in the mentioned statistic study. This proposal gives bond strenght values for each attack penetration (95% confidence interval), taking acount of the the actual residual stirrup section at anchorage length. Expressions for bond evaluation are included in F.2.3.

It is well known that the external pressure enhances the confinement of the bars at anchorage length, as it occurs at support regions. Consequently, it improves the bond strength and this positive effect is considered in Eurocode 2 when calculating ultimate bond stresses in sound ribbed bars[f.14].In order to explore this bond enhancement in corroded bars, tests were carried out with beams designed to fail by anchorage failure of the tensile bars (Figure f.7) and an empirical expression was obtained (see F.2.3)

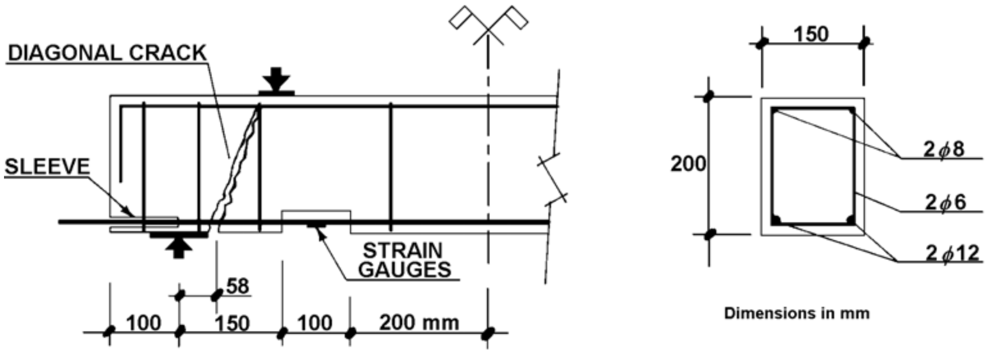


Figure F7 *Details of the beam and its failure at loading test.*

Figure f.8 shows the application of the mentioned proposals to an example of a 20 mm bar, without stirrups (curve 3) or with four 8 mm stirrups at the anchorage length. Curves 1 and 2 correspond to the bond strength reduction at end anchorage outside the support region and curve 4 to the bond strength reduction at support with an external pressure of 5 MPa. Homogeneous corrosion at stirrups is considered in curve 1 whereas pitting is considered in curve 2.

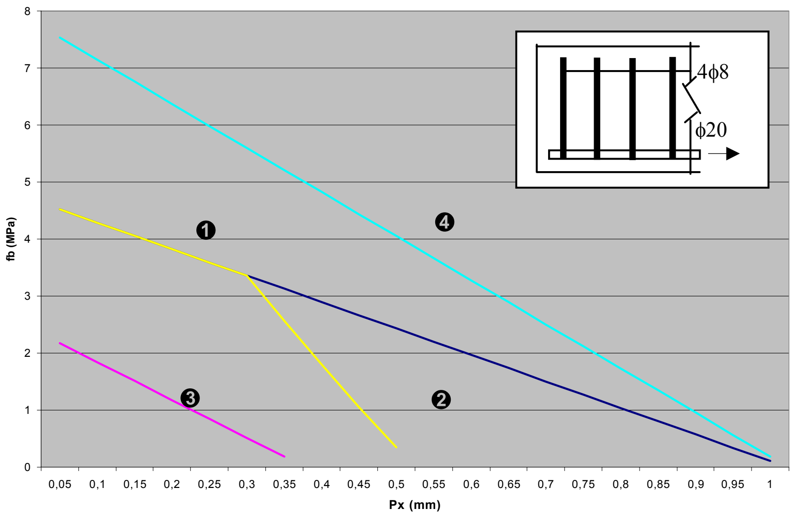


Figure F8. *Residual bond strength versus attack penetration*

<u>Composite action</u>

A hot topic for discussion of corroded beams is whether the loss of composite interaction between concrete and main reinforcement significantly affect their behaviour, assuming that the end bars were adequately anchored at support regions (beam effect versus arch effect) (Figure f.9).

According to the studies carried out by Cairns[f.15] in sound beams reinforced with unbonded bars, three main signs reflect the loss of composite action in beams with tensile reinforcement ratios higher than 1.5-2.0% submitted to shear force: a) reduction of load carrying capacity; b) reduction of ductility; and c) increase of concrete compressive strains in constant moment region (Figure f.10).

Although the first two signs were evident in the corroded tested beams[f.21], no increase of compressive strains was observed. Conversely, most of the tested beams with high tensile reinforcement ratio suffered premature concrete crushing for strain values significantly lower than 0.0035. Consequently, it could be stated that the loss of composite action, if it happened, did not have a relevant influence on the corroded beam behaviour. The reduction of both strength and ductility can be explained taking into account the reduction of steel section and the cracking of concrete cover.

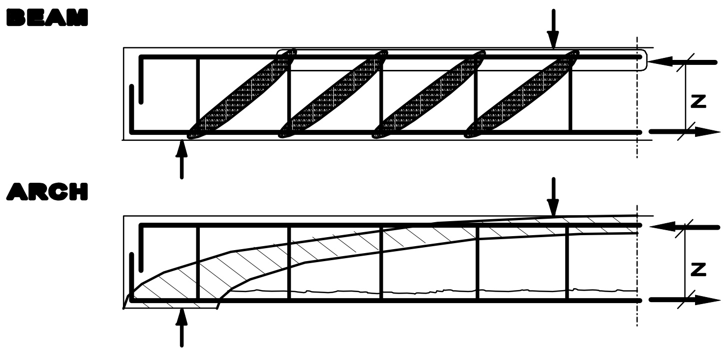


Figure F9 *Beam model (bonded reinforcement) and arch model (unbonded reinforcement).*

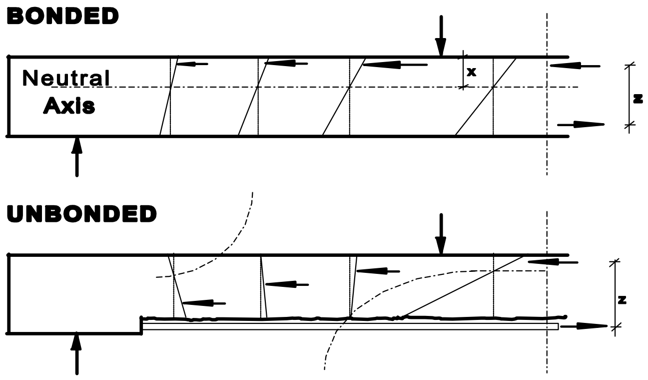


Figure F10 *Strain in concrete beams with either bonded or unbonded reinforcement.*

By the other hand, several studies carried out with slabs by Almusallam et al. showed that in slabs with corroded tensile reinforcement and without transversal reinforcement, the bond deterioration should be considered in the estimation of the load carrying capacity. It might represent the case in which a selective attack only affecting to tensile reinforcement occurs.

F.1.5. Estimation of the ultimate bending moment

An extensive work was done to evaluate the residual safety level and the serviceability performance of corroded beams [f.16,f.18,f.19,f.21,f.27]. Thirty beams of 2300 x 200 x 150 mm were tested (Figure f.11) with reinforcing bars corroded up to 600μm and heavy pitting in some cases.

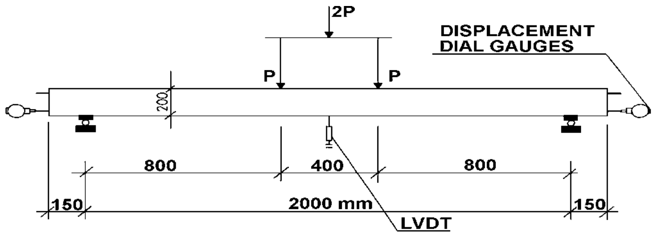


Figure F11 Detail of the beam and loading tests arrangement.

Beams with low ratio of tensile steel (0.5%) failed by the steel with significant concrete cracking at the tensile bottom part and keeping enough ductility behaviour in most of the cases[f.21], (Figure f.12). Conversely, beams with high ratio of tensile reinforcement (1.5%) and high amount of shear reinforcement (ϕ 6 mm stirrups at 85 mm spacing) failed by crushing of the concrete at the compression chord with buckling of the compression reinforcement (Figure f.13). This failure was accelerated by the deterioration of the concrete which surrounded the corroded compression reinforcement and it occurred with significant reduction of the ductility of the beam.

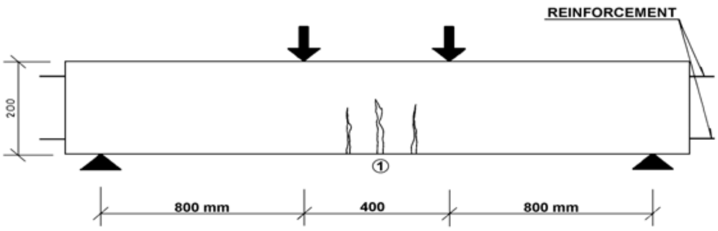


Figure F12 *Type of failure in beams with low tensile reinforcement ratio.*

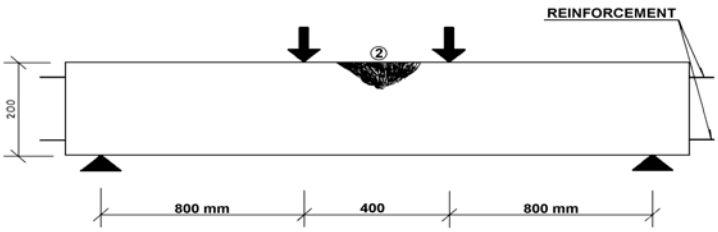


Figure F13 *Type of failure in beams with high tensile resinforcement ratio.*

Figure f.14 shows the experimental values of maximum bending moments in beams with low and high tensile reinforcement. It also includes the predicted values by using Eurocode 2, taking account of the residual tensile bar section and either the sound concrete section (higher value) or the deteriorated section without the contribution of the concrete cover (lower value).

Thus, a conservative value of the ultimate bending moment of these beams can be predicted by using the conventional concrete models, usually applied in non deteriorated concrete sections, and considering the reduced bar section and the reduced concrete section without the contribution of the concrete cover at the compression chord.

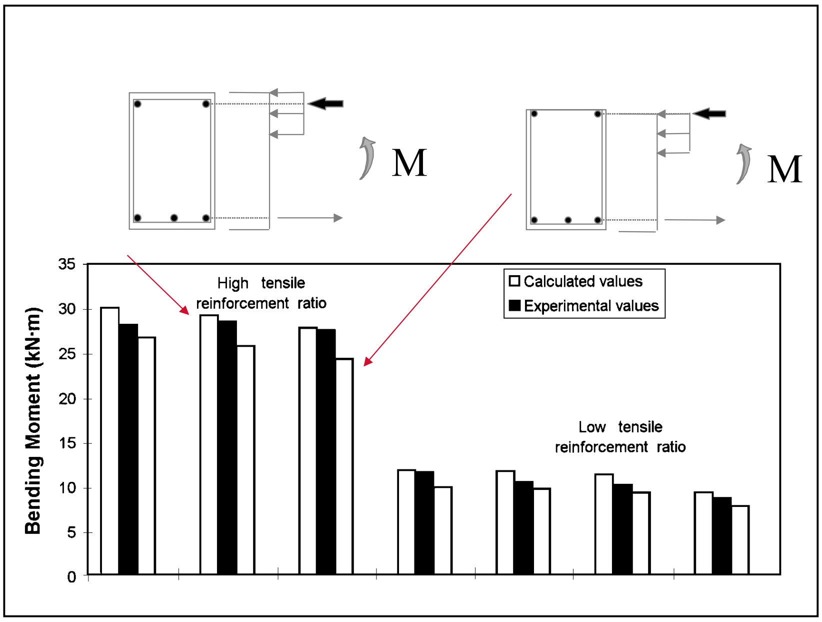


Figure F14 Ultimate bending moment in beams with corroded reinforcement.

F.1.6. Estimation of the ultimate shear force

Sound beams with high ratio of tensile reinforcement but usual amount of transverse reinforcement are designed to fail by bending moment (compresion concrete). However, corrosion affected the type of failure and most of corroded beams tested in the research work failed by shear[f.16,f.17,f.18]. The causes of this behaviour were:

a) the pitting corrosion at the stirrups
b) the spalling of compression concrete cover produced by the corrosion at both the compression bars and the stirrups in addition to the shear stresses
c) the spalling of the side cover produced by the corrosion at stirrups.

The results of the experimental work were checked by the standard method included in ENV1992-1. Figures f.15 and f.16 show the type of failure which was obtained in the tested beams due to either shear or shear combined with bond.

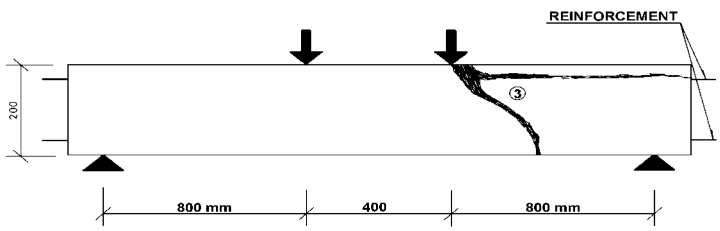


Figure F15 Shear failure.

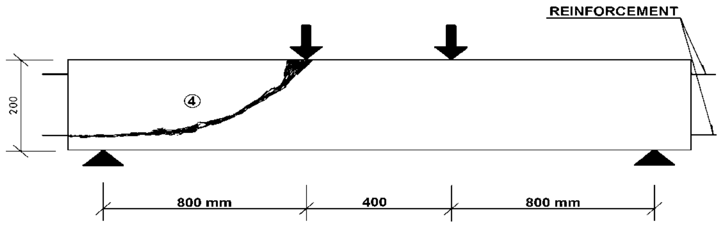


Figure F16 *Shear and bond failure.*

Figure f.17 show the experimental maximum shear force in some corroded beams with high tensile reinforcement ratio (1.5%) and stirrups ϕ 6 mm at 170 mm spacing. It also includes the predicted values by the standard method in Eurocode 2, considering the residual bar section and either the sound concrete section (higher value) or the deteriorated concrete section without the concrete cover at the top compression chord (intermediate value) or the deteriorated concrete section without the concrete cover at both the compression chord and the sides.

Thus, a conservative value of the ultimate shear force can be predicted by using the conventional concrete models and considering the reduced reinforcing bar section and the reduced concrete section without the contribution of concrete cover at both the compression chord and the sides.

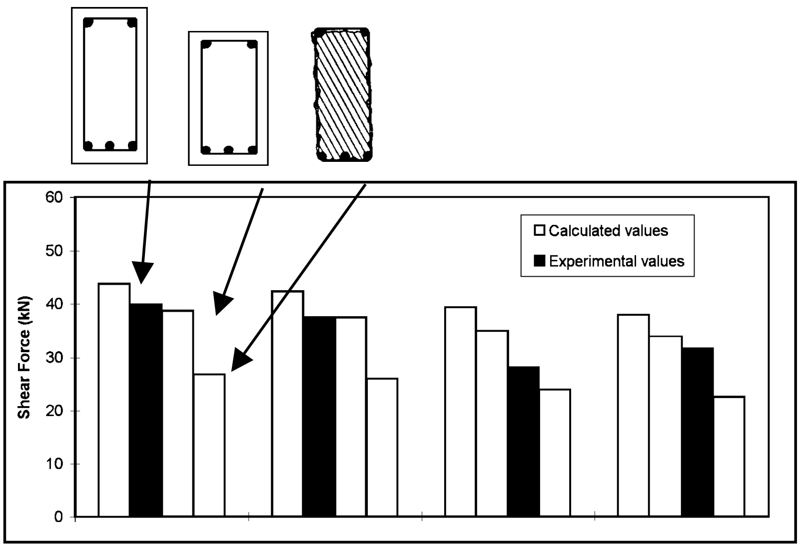


Figure F17 *Ultimate shear force in beams with corrode reinforcement.*

F.1.7. Estimation of the ultimate axial force/bending moment

An experimental study was carried out using 24 concrete elements of 2000 x 200 x 200 mm, to establish the relationship between the level of corrosion and the structural performance of deteriorated concrete columns. Reinforced concrete columns without corrosion and with different levels of corrosion were tested and different reinforcing details were considered[f.17,f.20]. Columns ends were designed to avoid failure during the loading testing.

Although the columns were axially loaded, some eccentricities were detected during the loading test, due to the test conditions, the geometric imperfections of the columns and the corrosion. The column failure was generally initiated by the cracking and spalling of the concrete cover and the failure of one or more links, which were heavily affected by localized corrosion (pitting), and provoked the buckling of main bars under the applied load.

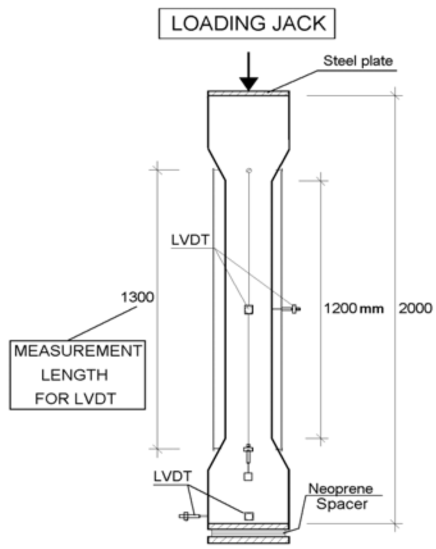


Figure F18 *Loading test arrangement of the columns.*

Three main aspects seem to affect the the behaviour of the corroded columns: the deterioration of the concrete section, the increase of the load eccentricity due to assimetric deterioration of the concrete cover and the likely reduction of reinforcement strength due to premature buckling. Figure f.19 shows the experimental values of the axial force in some of the tested columns. It also shows the calculated values using the conventional models but considering the reduced bar section, either the complete or the reduced concrete section (without the concrete cover at the four sides) and two values of eccentricity: 0 mm and 20 mm.

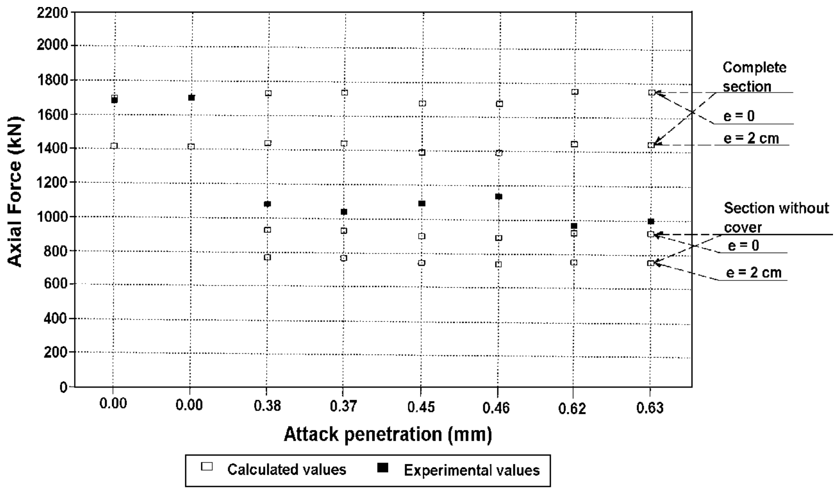


Figure F19 *Axial force values in tested columns.*

Figure f.20 represents the maximum experimental loads in corroded columns with their moments (obtained from the eccentricities). As biaxial eccentricities were produced in the tests, the values of the bending moments in these figures were obtained by applying a simplified method which reduces the biaxial eccentricities to an equivalent uniaxial one.

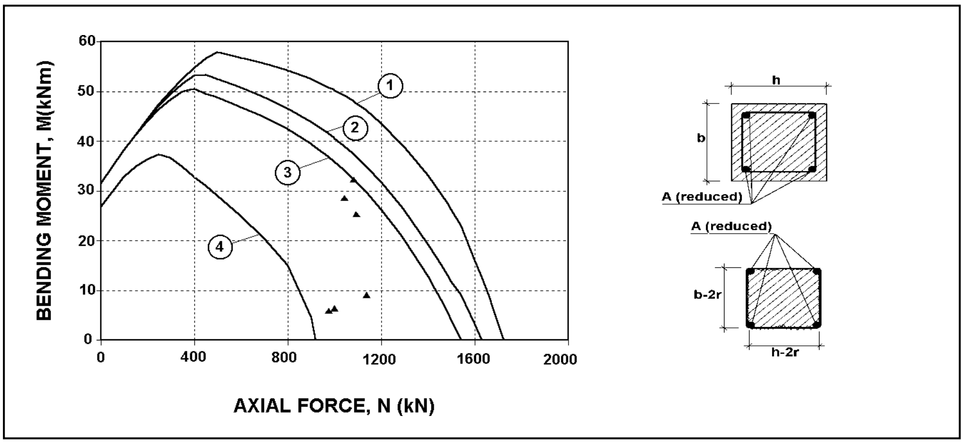


Figure F20. *Experimental and calculated values (Nu , Mu) in corroded columns*

Figure f.20 also shows the curves for the different pairs of calculated ultimate values (N_u, M_u). Whereas in curves 1, 2 and 3, the non deteriorated concrete section was considered, curve 4 corresponds to a deteriorated section without the concrete cover on the four sides. Finally, curves 2 and 3 were obtained assuming that two or three consecutive links failed due to corrosion and, consequently, the stress of compresion bars were reduced[f.20]. The experimental values are placed between those obtained from the non deteriorated and the deteriorated concrete sections.

The spalling was mainly initiated on one side of some tested columns with corroded reinforcement. Therefore, Figure f.21 represents the curves for different pairs of calculated ultimate values (N_u , M_u) for a concrete section without the cover on one side. Whereas curve 1 does not take account of likely buckling of compresion bars, curves 2 and 3 consider the failure of two or three consecutive links and consequently, the stress of compression bars was reduced (Figure f.20). Figure f.21 shows that some experimental values can be predicted when considering this type of deteriorated concrete section together with the premature buckling of the main bars.

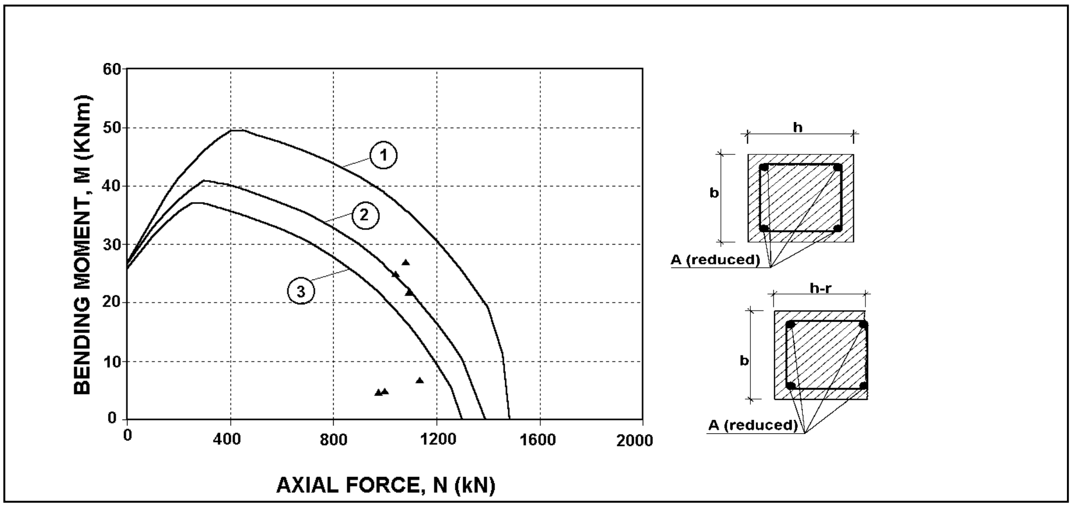


Figure F21. *Experimental and calculated values (Nu , Mu) in corroded columns*

F.2. EVALUATION OF THE MAIN EFFECTS OF CORROSION

F.2.1. Reinforcement

The attack penetration can be obtained through the following expression:

$$P_x = 0.0115\ I_{corr}^{rep}\ t \qquad [f.1]$$

P_x	is the average value of the attack penetration (decrease of bar radius), in mm.
I_{corr}^{rep}	is the representative value of the corrosion rate during the time t, in $\mu A/cm^2$
t	is the time elapsed since the aggressive reached the bar (propagation period), in years.

The residual cross sectional area of the corroded bar can be estimated through the following expression for the diameter :

$$\phi_t = \phi_0 - \alpha\ P_x \qquad [f.2]$$

where

ϕ_t	is the residual diameter at time t, in mm
ϕ_0	is the nominal diameter, in mm
α	is equal to 2 (carbonated concrete) and it is up to 10 at the pits (chloride contaminated concrete)[f.1].
P_x	is the average value of the attack penetration (decrease of bar radius), in mm.

When expression [f.2] is applied to the decrease of the diameter at the pits, the homogenous decrease of the diameter, produced at the same time, has not been taken into account because it can be considered as negligible in comparison with the pitting.

These residual diameter values ϕ_t corresponding to the present state of the structure might be contrasted by direct geometric measurements at some spots in order to establish and to correct the accuracy of calculated values from both I_{corr}^{rep} and t values.

The suggested maximum values for α correspond to a single bar. However, these maximum values might be reduced when the estimation of the decrease of the diameter of several bars (> 3) at the same cross section of a beam or column is needed, unless a significant loss of bond has been produced.

It is worth pointing out that bending redistribution should be taken with caution due to the loss of ductility in corroded bars.

F.2.2. Concrete cracking

The empirical expression developed from the experimental results to assess cracking conditions in corroded structures[f.8] and to obtain the characteristic value of the crack width is the following:

$$w = 0.05 + ß\ [P_x - P_{x\,o}]\ [w \leq 1.0 \text{ mm}] \qquad [f.3]$$

where w is the estimated crack width in mm, P_x is the attack penetration (bar radius decrease) in mm, $P_{x\,o}$ is the attack penetration corresponding to cracking initiation and ß is a coefficient which depends on the position of the bar [ß=10 for top cast bars and 12.5 for bottom cast bars]. The attack penetration $P_{x\,o}$ needed for the cracking initiation can be estimated by the following expression:

$P_{x\,o} = (83.8 + 7.4\ c/\phi\ - 22.6\ f_{c,sp}).10^{-3}\ \ [P_{x\,o} \geq 0]$ [f.4]

where P_{xo} is the attack penetration in mm, c/ϕ is the cover/diameter ratio and $f_{c,sp}$ is the splitting tensile strength in Mpa, which can be obtained through testing or by the following expressions of Eurocode 2:

$f_{ct} = 0.3\ f_{ck}^{2/3}\quad f_{ck}\ \leq 50\ N/mm^2$ [f.5]

$f_{ct,sp} = f_{ct}/0.9 = 0.333\ f_{ck}^{2/3}$ [f.6]

F.2.3. Bond

Expressions developed to determine how attack penetration affects bond deterioration are the following:

If $\rho > 0.25$ $f_b = 4.75 - 4.64\ P_x$ [f.7]

If $\rho < 0.25$ $f_b = 10.04 + [-6.62 + 1.98\ (\rho/0.25)]\ [1.14 + P_x]$ [f.8]

$\rho = n\ [(\phi_w - \alpha\ P_{x\,w}) / \phi]^2$

where:

P_x	is the average value of attack penetration at main bars, in mm.
$P_{x\,w}$	is the average value of attack penetration at transverse bars, in mm
ϕ	is the diameter of the main bar, in mm.
ϕ_w	is the diameter of the transverse bar, in mm.
α	2 (homogenous corrosion at transverse reinforcement) $\leq$ 10 (pitting at transverse reinforcement)
n	is the number of transverse bars at anchorage length of the main bar

If there are not stirrups, the residual bond strength can be estimated by the following expression:

$f_b = 2.50 - 6.62\ P_x$ [f.9]

When bars are anchored at support zones where an external pressure p confines the bar, the residual bond strength can be estimated by:

$f_b = (4.75 - 4.64\ P_x) / (1-0.08\ p)$ [f.10]

where p is the external pressure in MPa.

All formula described above were obtained through experimental tests carried out with ribbed bars specimens. When the assessed structure is made of plain bars, a reduction of f_b is proposed according to EC2 criteria[1]. This criteria determines a ratio between plain and ribbed bars of 1/2.25, so for plain bars anchored outside the support zones, the residual bond strength could be estimated by the following expressions:

If $\rho > 0.25$ $f_b = (4.75 - 4.64\ P_x)/2.25$ [f.11]

[1] **Note :** *expressions for corroded plain bars have not been calibrated by testing.*

If $\rho < 0.25$ $f_b = (10.04 + [-6.62 + 1.98 (\rho/0.25)] [1.14 + P_x])/2.25$ [f.12]

$\rho = n [(\phi_w - \phi P_{x\,w}) / \phi]^2$

When no stirrups are present, the residual bond strength can be estimated by the following expression:

$f_b = (2.50 - 6.62 P_x)/2.25$ [f.13]

When bars are anchored at support zones where an external pressure p confines the bar.:

$f_b = (4.75 - 4.64 P_x) / (2.25(1-0.08 p))$ [f.14]

F.3. ULTIMATE LIMIT STATES

F.3.1. Ultimate bond strength

- The verification of ultimate bond strength at support zones should be carried out according to the expressions included in F.2.3.
- The verification of bond at intermediate parts of the reinforcing bars should be carried out as follows:

The bond stress between the bars and the surronded concrete due to the actions can be estimated by:

$V_d / 0.9\, d\; n'\; \pi\; \phi$ [f.15]

where: V_d is the shear force
d is the effective depth (or the reduced effective depth)
n' No. of tensile reinforcing bars with diameter ϕ

The residual bond strength between steel and concrete at intermediate parts of the bar can be estimated by the expressions [f.9], [f.10], [f.11],[f.12], [f.13] and [f.14] where

$\rho = 200 [(\phi_w - \alpha P_{x\,w}) / \phi]^2 / s$

and s is the spacing of transverse reinforcement, in mm.

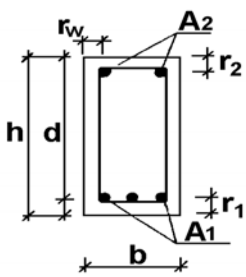
When bond stress is higher than the value obtained through [f.9], [f.10], [f.11], [f.12], [f.13] and [f.14], the composite effect is lost causing some reduction of the ultimate load when the failure is due to bending moment. Conversely, shear failure occurs very seldom within the length of the beam over which the bond is lost, as vertical loads are equilibrated by the vertical component of an inclined compressive strut[f.15].

An increase of the deflections, crack spacing and crack width is also produced due to some loss of composite action

F.3.2. Ultimate bending moment

An estimation of the residual ultimate bending moment can be obtained by the Eurocode 2, with some modifications depending on the following parameters:

- Attack penetration $P_{x\,1}$ at tensile bars
- Attack penetration $P_{x\,2}$ at compression bars
- Attack penetration $P_{x\,w}$ at shear reinforcement
- Ratio of tensile bars ρ_1
- Ratio of compression bars ρ_2
- A_α : cross sectional area of shear reinforcement per length unit of beam
- Anchorage conditions of tensile bars



$\rho_1 = A_1/(b\ d)$ tensile reinforcement
$\rho_2 = A_2/(b\ d)$ compression reinforcement
s spacing of shear reinforcement

Figure f.22 *Notation*

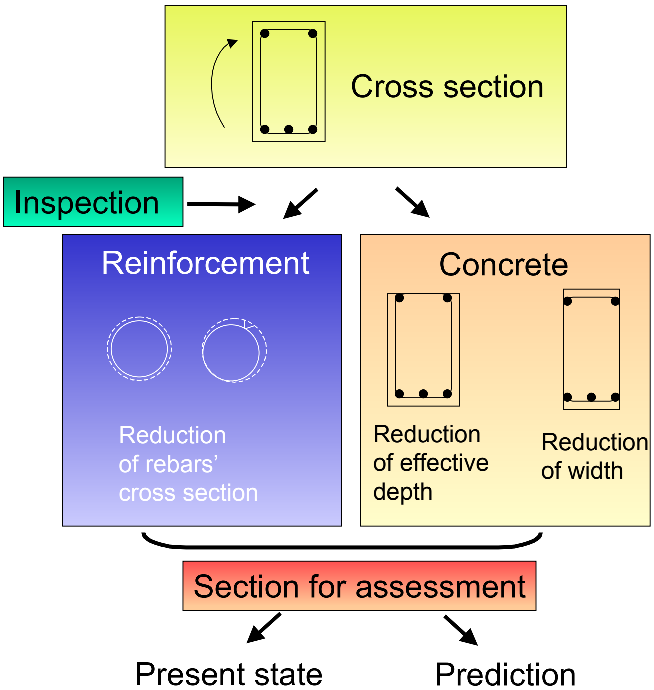


Figure f.23. *Estimation of the ultimate bending moment*

As it is shown in figure f.23, the geometrical data to be considered as an input in the calculation of the ultimate bending moment will be the corresponding to the RC cross section with the following exceptions:

a) Reinforcement

The reduced cross sectional area of the rebars has to be obtained from the value of attack penetration, according to section F.2.1.

b) Concrete

Prediction/assessment of the future state :

Beams

a) Reduced effective depth $d-r_2$ instead of d if:

- Ductile failure: $\rho_1 < 1.0$ %

  $\rho_2 < 0.5$ % and ($P_{x\,2}$ or $P_{x\,w}$) > 0.4 mm

  $\rho_2 > 0.5$ % and ($P_{x\,2}$ or $P_{x\,w}$) > 0.2 mm
  (risk of spalling provoked by corrosion of compression/shear reinforcement)

-Brittle failure: $\rho_1 > 1.5$ %

  $\rho_2 > 0.5$ % and ($P_{x\,2}$ or $P_{x\,w}$) > 0.2 mm
  (risk of spalling provoked by corrosion of compression/shear reinforcement)

  $\rho_2 > 0.5$%, $A_\alpha > 0.0018$ b and ($P_{x\,2}$ or $P_{x\,w}$) > 0.1 mm
  (risk of spalling provoked by both shear stress and corrosion of compression reinforcement).

b) Reduced width $b-2r_w$ instead of b if:

  $\rho_1 > 1.5$ % , $A_\alpha > 0.0036$ b and ($P_{x\,2}$ or $P_{x\,w}$) > 0.2 mm
  (risk of spalling provoked by both shear stress and corrosion of compression/shear reinforcement).

For intermediate values of ρ_1, values for geometrical data between those previously suggested can be taken into account.

When the assessment of the future state is carried out, the evolution of the concrete cover cracking and spalling must be taken into account. The starting point for loosening of concrete cover can be considered when the attack penetration at compression bars $P_{x\,2}$ reaches the needed value for cracking initiation $P_{x\,0}$ as it is explained in F.2.2:

$$P_{x\,o} = (83.8 + 7.4\ c/\phi\ - 22.6\ f_{c,sp}).10^{-3}\quad [P_{x\,o} \geq 0] \qquad \text{[f.4]}$$

Once this value has been reached, it is not well known how the load bearing capacity evolves till the concrete cover spalls. A linear interpolation can be assumed between these two points to cover all intermediate cases[2].The same criterion has been adopted between cover spalling at compression zone and lateral spalling.

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[2] **Note :** *no experimental testing has been carried out regarding the evolution of load bearing capacity between*

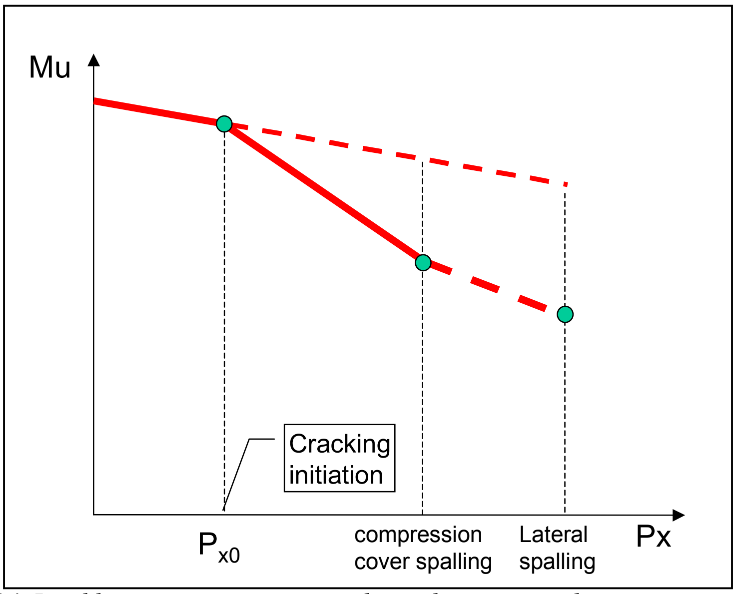

Figure F24. *Load bearing capacity extrapolation between cracking initiation and spalling*

Solid slabs

When solid slabs are assessed, the evaluation of ultimate bending moment, the reduced width has no sense. Furthermore, slabs have always ductile failure ($\rho_1 < 1$ %), so the effective depth should be taken as d-r_2 instead of d if:

$\rho_2 < 0.5$ % and $P_{x\,2} > 0.4$ mm

$\rho_2 > 0.5$ % and $P_{x\,2} > 0.2$ mm

Slabs will be assessed as beams of as solid slabs depending on engineering criteria.

<u>Evaluation of the present state</u>:

The use of the reduced effective depth and width can be considered according to the previously mentioned criteria. However, it shall also be taken into account the results obtained in the inspection (cover delamination, cracking, spalling, ...).

F.3.3 Ultimate shear force

The results of the experimental work in which this manual is based on were checked by the standard method included in ENV1992-1. This method is of application when deteriorated structures are assessed. Altenately, expressions included in prEN 1992-1 as the variable strut method can be used for the assessment of not heavily corroded elements.

An estimation of the residual ultimate shear force could be obtained by means of the standard method established in Eurocode 2 with some modifications to consider the effects of corrosion on the section's geometry and the deteriorarion of mechanical characteristics on shear capacity. Parameters to be taken into account are the following:

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cracking initiation and spalling.

- Attack penetration $P_{x\,1}$ at tensile bars
- Attack penetration $P_{x\,2}$ at compression bars
- Attack penetration $P_{x\,w}$ at shear reinforcement
- Ratio of tensile bars ρ_1
- Ratio of compression bars ρ_2
- Spacing of shear reinforcement s
- A_α : cross sectional area of shear reinforcement per length unit of beam

According to EC2, the design value for the shear capacity is given by the following expressions:

a) for members not subjected to axial forces, with vertical shear reinforcement, the shear capacity is the smaller value of:

$$V_{Rd,sy} = \frac{A_{sw}}{s} z\, f_{ywd} \cot\theta \qquad [f.16]$$

$$V_{Rd,max} = \frac{b_w\, z\, \nu\, f_{cd}}{\cot\theta + \tan\theta} \qquad [f.17]$$

with

$$\frac{A_{sw}\, f_{ywd}}{b_w\, s} \le \frac{1}{2} \nu\, f_{cd}$$

where $\nu = 0{,}7 - \frac{f_{ck}}{200} \ge 0{,}5$

b) for members not subjected to axial forces, with inclined shear reinforcement, the shear capacity is the smaller value of:

$$V_{Rd,sy} = \frac{A_{sw}}{s} z\, f_{ywd}\, (\cot\theta + \cot\alpha) \sin\alpha \qquad [f.18]$$

$$V_{Rd,max} = b_w\, z\, \nu\, f_{cd} \frac{\cot\theta + \cot\alpha}{1 + \cot^2\theta} \qquad [f.19]$$

with $\frac{A_{sw} f_{ywd}}{b_w s} \le \frac{0{,}5\, \nu\, f_{cd} \sin\alpha}{1 - \cos\alpha}$

When applying these expressions in both beams and slabs, it is suggested to consider the influence of bond deterioration in the contribution of tensile reinforcement to shear capacity. It is suggested to use $\rho_1(f_b/f_{bo})$ instead of ρ_1 in the formula [f_{bo}, bond strength in sound reinforcement (x=0); f_b, residual bond strength due to corrosion with attack penetration P_x]

Values around θ=45 are suggested. Lower θ values should be avoided because bond stresses in longitudinal tensile stresses may not be resisted due to bond reduction

The geometrical data to be considered as an input in the calculation of shear capacity will be the corresponding to the RC cross section with the following exceptions:

a) Reinforcement

The reduced cross sectional area of the bars has to be obtained from the value of attack penetration, according to section F.2.1. It is worth pointing out that <u>a conservative value of the residual cross sectional area of the links should be considered</u>, because:

- Links have small diameter (6 to 12 mm, usually) and, consequently, they are very sensitive to corrosion.

- Transverse reinforcement has a great influence in the estimation of the residual shear force value and, consequently, to the potential brittle failure of the RC element.

Reduction of the contribution of tensile reinforcement to shear resistance should be considered due to bond strength reduction. Consequently, reinforcement ratio ρ_1 should be reduced depending on the level of residual bond strength.

b) Concrete

Prediction/assessment of the future state:

Beams

Reduced effective depth $d\text{-}r_2$ instead of d if:

- $A_\alpha < 0.0018\ b$

  $s > 0.6\ d$ and $(P_{x\,2}$ or $P_{x\,w}) > 0.2$ mm
  (a conservative value of ultimate shear force is suggested due to the critical influence of corroded shear reinforcing highly spaced)

  or

  $\rho_2 > 0.5\%$ and $(P_{x\,2}$ or $P_{x\,w}) > 0.2$ mm
  (risk of spalling provoked by corrosion of compression/shear reinforcement)

- $A_\alpha > 0.0018\ b$

  $\rho_2 < 0.5\ \%$ and $P_{x\,w} > 0.2$ mm
  (risk of spalling provoked by both shear stress and corrosion of shear reinforcement).

  $\rho_2 > 0.5\%$ and $(P_{x\,2}$ or $P_{x\,w}) > 0.1$ mm
  (risk of spalling provoked by both shear stress and corrosion of compression/shear reinforcement).

Reduced width $b\text{-}2r_w$ instead of b if:

- $A_\alpha > 0.0018\ b$

  $s > 0.6\ d$ and $(P_{x\,2}$ or $P_{x\,w}) > 0.4$ mm
  (risk of spalling provoked by both shear stress and corrosion of compression/shear reinforcement).

  $s < 0.6\ d$ and $(P_{x\,2}$ or $P_{x\,w}) > 0.3$ mm
  (risk of spalling provoked by both shear stress and corrosion of compression/shear reinforcement).

When the assessment of the future state is carried out, the evolution of the concrete cover cracking and spalling must be taken into account. The starting point for loosening of concrete cover can be considered when the attack penetration at compression bars $P_{x\,2}$ reaches the needed value for cracking initiation $P_{x\,0}$ as it is explained in F.2.2

$$P_{x\,o} = (83.8 + 7.4\ c/\phi\ - 22.6\ f_{c,sp}).10^{-3}\quad [P_{x\,o} \geq 0] \qquad [f.4]$$

Once this value has been reached, it is not well known how the shear capacity evolve till the concrete cover spalls. A linear interpolation can be assumed between these two points to cover all intermediate cases[3].

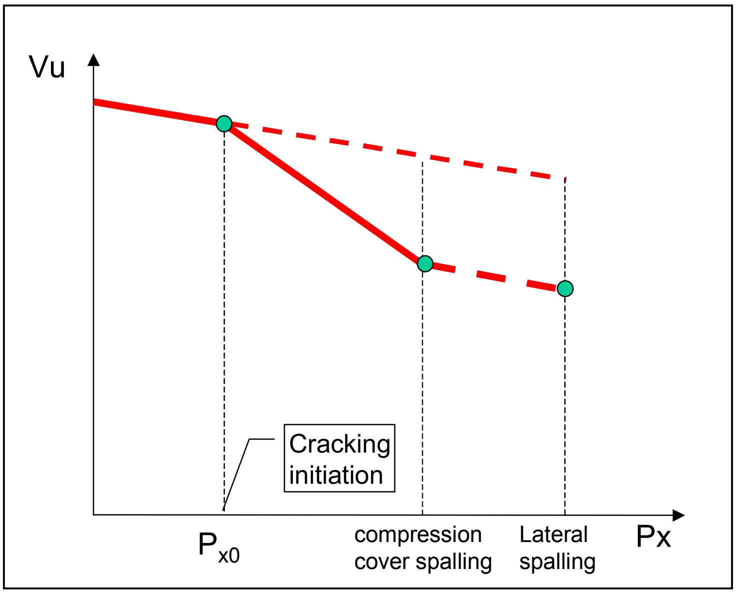

Figure f.25. Evolution of shear capacity

Solid slabs

For members without transverse reinforcement, the following formula should be used to evaluate V_{u2}

$$V_{u2} = 0.12\ [1+(200/d)^{1/2}]\ [100\ \rho_1\ f_{ck}]^{1/3} \qquad [f.20]$$

For these elements, effective depth should be taken as d-r_2 if:

- $\rho_2 < 0.5$ and $P_{x\,2} > 0.4$ mm
- $\rho_2 > 0.5$ and $P_{x\,2} > 0.2$ mm

The reduction of the contribution of tensile reinforcement to shear resistance should be considered due to bond strength reduction. Consequently, reinforcement ratio ρ_1 should be reduced depending on the level of residual bond strength.

It is suggested to use $\rho_1(f_b/f_{bo})$ instead of ρ_1 in the formula [f_{bo}, bond strength in sound reinforcement (x=0); f_b, residual bond strength due to corrosion with attack penetration P_x].

Slabs will be assessed as beams of as solid slabs depending on engineering criteria.

<u>Evaluation of the present state</u>:

The use of the reduced effective depth and width can be considered according to the previously mentioned criteria. However, it shall also be taken into account the results obtained in the inspection (cover delamination, cracking, spalling, ...)

[3] **Note :** *no experimental testing has been carried out regarding the evolution of load bearing capacity between cracking initiation and spalling.*

F.3.4. Punching

Punching has to be considered in slabs on isolated columns or under concentrated loads with the following conditions[4]:

Slab/column without shear reinforcement:

$$v_{Ed} \leq V_{rd,c} \qquad [f.21]$$

$$v_{Ed} = V_{Ed} / (u\ d) \qquad [f.22]$$

$$V_{rd,c} = 0.12\ [1 + (200/d)^{0.5}]\ [100\ \rho_1\ f_{ck}]^{1/3} \quad \text{with } \rho_1 = (\rho_{lx}\ \rho_{ly})^{1/2} \qquad [f.23]$$

- ✓ Reinforcement ratio ρ_{lx} , ρ_{ly} should be reduced depending on the level of residual bond strength, as it was exposed for beams. It is suggested to use $\rho_1(f_b/f_{bo})$ instead of ρ_1 in the formula [f_{bo}, bond strength in sound reinforcement (x=0); f_b, residual bond strength due to corrosion with attack penetration P_x].

- ✓ Effective depth should be taken as d-r_2 if
  - $\rho_2 < 0.5$ and $P_{x\,2} > 0.4$ mm
  - $\rho_2 > 0.5$ and $P_{x\,2} > 0.2$ mm

Slabs/columns with shear reinforcement

When slabs or columns with shear reinforcement are assessed, considerations made for beams should be taken into account. According to EC2, the punching shear capacity of slabs with punching shear reinforcement should be verified in three zones:

- the zone inmediately adjacent to the column or loaded area

$$v_{Ed} = \frac{V_{Ed}}{u\,d} \leq 0{,}5\ \nu\ f_{cd} \qquad [f.24]$$

- the zone in which the shear reinforcement is placed

$$v_{Rd,cs} = 0{,}75\ v_{Rd,c} + \sum A_{sw}\ f_{yd}\ \sin\alpha\ /\ u\,d \qquad [f.25]$$

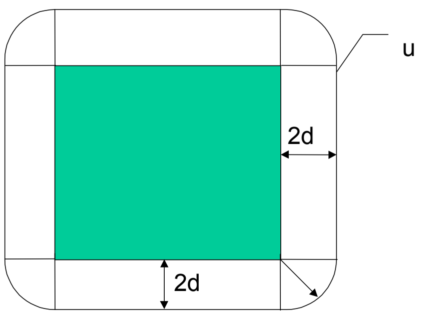

Figure F26. Critical perimeter

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[4] **Note** : *no experimental work regarding punching failure of corroded elements has been carried out. Suggested values have been extrapolated from the experience gained from shear failure of corroded beams.*

F.3.5. Estimated ultimate axial force/bending moment

An estimation of the residual ultimate axial force in a column could be obtained by the Eurocode 2, with some modifications depending on the following parameters:

- Attack penetration P_x at main reinforcement
- Attack penetration $P_{x\,w}$ at links
- Ratio of main reinforcement bars ρ
- Diameter of main reinforcement
- Diameter and spacing of links

The data to be considered as an input in the calculation of the ultimate axial force will be the corresponding to the reinforced concrete column with the following exceptions:

Additional eccentricity e_{corr}

Eurocode 2 considers eccentricity values of the applied load in non corroded columns:

- First order eccentricity $e_1 = M_d/N_d$
- Additional eccentricity due to geometrical imperfections e_a
- Second order eccentricity e_2

An additional mechanical eccentricity e_{corr} has to be added to these values, due to the likely different deterioration at each cover side of the concrete section cause by corrosion of the main bar and the links. According to the experimental research work, a conservative value $e_{corr} = r$ in both directions has to be added.

Reinforcement

- Reduction of cross section area:

    It shall be obtained from the attack penetration Px. It is worth pointing out that a conservative value of <u>the residual cross sectional area in the links should be considered, specially when pitting may occurs, because their influence on the buckling of the main bars at the ultimate</u>.

- Reduced strength at main reinforcing bars due to premature buckling:

    The theoretical stress values corresponding to the critical load of the main bars may be calculated by the Euler theory, considering "0.75 s" as the effective height of the main bars for the estimation of the slenderness ratio, being "s" the link spacing. *[To assume 0.5 s as the effective length would have been unsafe because the deteriorated links enabled the bars to exhibit some rotation at the links].*

The design stress of the steel f_{yd} is lower than the Euler critical stress, for the maximum links spacing specified in the Codes, which ranges between 10 and 15 times the diameter of the main. However, this spacing increases in deteriorated columns when some links fail by localized corrosion.

A conservative value of the strength at the main compression bars affected by the premature buckling can be estimated by the following expression:

$$\sigma_s = k\ f_y\ f_{yd} \qquad [f.26]$$

where: f_y is the yield stress, k is 1.0 for non corroded columns; 0.5 for corroded columns, when one link fails; 0.2 for corroded columns, when two consecutive links fail; and 0 for corroded columns, when more than two consecutive links fail, f_{yd} is the design stress

It is worth pointing out that not only the failure of the links but also the spalling of the concrete cover can induces the buckling of the reinforcing bars, mainly when type 1 links are used instead of type 2 (Figure f.28).

Concrete

Prediction/assessment of the future state :

The reduced dimensions “a-2r”, “b-2r” shall be considered instead of “a”, “b” if:

Figure F27 *Detail of links.*

$\rho \leq 0.01$ and $\phi \leq 16$ mm and $P_x \geq 0.2$ mm

or

$\rho \leq 0.01$ and $\phi \leq 20$ mm and $P_x \geq 0.1$ mm

or

$\rho \geq 0.02$ and $P_x \geq 0.1$ mm

In some cases, delamination can mainly be produced in one column side, depending of the reinforcement detailing, exposition to the aggressive, etc. Then, a reduced concrete section without one cover side shall be considered.

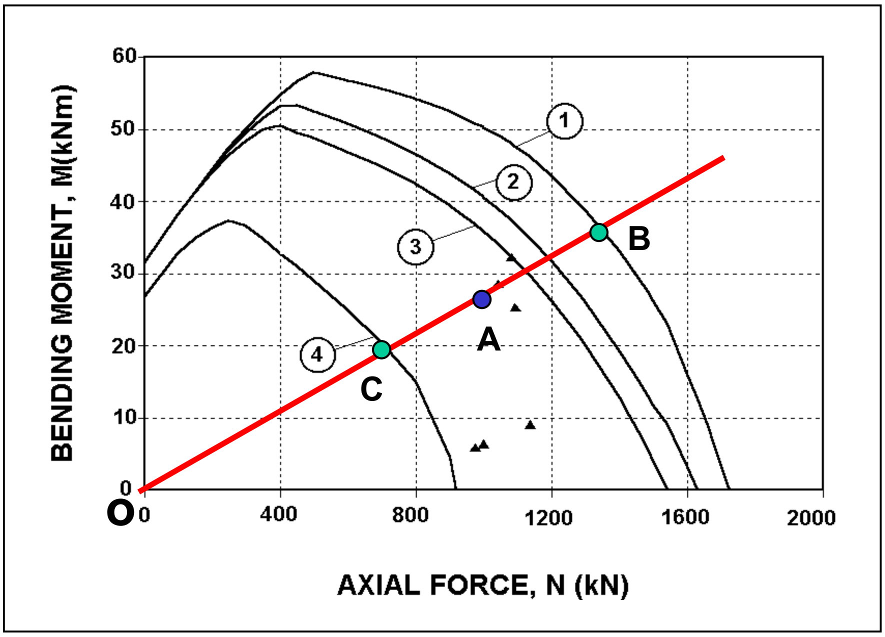

Figure F28. *Residual service life for (N,M) affected sections*

As it is shown in figure f.28, once the eccentricity line has been obtained through the relationship M_d/N_d and the considerations for corroded columns (additional eccentricity due to the different deterioration at each side of the support), points of instersection B and C with N_u-M_u curves correponding to a) curve of non-deteriorated section ① and b) curve of maximum deterioration ④ respectively, two different levels of deterioration can be determined.

As (N_d,M_d) values are known, point A is also determined. The safety factors related to these two deterioration steps can be obtained comparing the values of (OB, OA) and (OC, OA) respectively.

This comparison can be carried out through the values of Na, Nb and Nc or through the values of Ma, Mb, Mc depending on the location of the assessed section. For instance, axial force comparisons should be chosen when columns located at intermediate or low plants of buildings are assessed, whereas the safety margen of columns located at a roof floor should be determined through bending moment comparison.

Intermediate points of the eccentricity line could be determined through linear extrapolation from known (N-M) points.

<u>Evaluation of the present state</u>:

The use of reduced geometrical dimensions "a-2r" and "b-2r" can be considered according to the previous mentioned criteria. However, it shall also be taken into account the results obtained in the inspection (cover delamination, cracking, spalling, ...).

F.4 SERVICEABILITY LIMIT STATES

SLS have mainly to be considered when dealing with the prediction of the future state of the structure. The opinion of the owner shall also be considered.

Two main aspects affected the behaviour of the corroded beams at Serviceability Limit States (SLS): the deterioration of the materials and some loss of composite effect due to the bond deterioration. Although the loss of composite effect was not considered at Ultimate Limite States (ULS) because other factors as pitting at stirrups and heavy deterioration of concrete section were more relevant on the beam performance, this loss needs to be considered when predicting the beam behaviour at SLS.

A rough approach to both deflection of the beam at midspan and the crack width produced by the loads, at right angle to the main to the main tensile bars, can be obtained by using Eurocode 2 models with the reduced steel section but considering the reduced bond characteristics of the ribbed bars as those of the sound plain bars. In heavily corroded beams, the deteriorated concrete section without the compression concrete cover should also be considered.

F.4.1 Deflection

Deflections may be obtained using chapter 7.4 of Eurocode 2 with some modifications, depending on the following parameters:

- Attack penetration $P_{x\,1}$ (tensile bars) at mid span
- Average value of attack penetration $P_{x\,2}$ (compression bars)
- Ratio of compression bars ρ_2
- f_{bo}, bond strength in sound reinforcement (P_x=0)
- f_b, residual bond strength due to corrosion with attack penetration P_x

The data to be considered as an input in the calculation of the deflection will be the corresponding to the reinforced concrete beam with the following exceptions:

- Reinforcement: the reduced cross sectional area of the rebars at the mid span zone of the beam, obtained from the values of attack penetration $P_{x\,1}$ and $P_{x\,2}$, without considering the effect of pitting.

- Effective depth of the beam d- r_2 instead of d if:

  - $\rho_2 < 0.5\%$ and $P_{x\,2} > 0.40$ mm
  - $\rho_2 > 0.5\%$ and $P_{x\,2} > 0.20$ mm

  When intermediate cases between $P_{x\,o}$ and $P_{x\,2}$ are assessed, a linear interpolation is suggested to determine deflection.

- k : coefficient to take into account the bond properties of reinforcement as the relationship between the residual bond strength and bond strength in sound reinforcement ($k = f_b/f_{bo}$) [f_{bo}, bond strength in sound reinforcement (P_x=0); f_b, residual bond strength due to corrosion with attack penetration P_x].Corrosion affects bond strength and, consequently, coefficient k has to be included into the actual expressions of Eurocode 2 as follows :

$$\zeta = 1 - k\,\beta_1 \left(\frac{\sigma_{sr}}{\sigma_s}\right)^2 \qquad k \geq 0{,}5 \qquad [f.27]$$

F.4.2 Crack width w

Cracking in corroded concrete structures is produced by the formation of rust which generates tensile splitting stresses in the concrete cover around a corroded bar. Thus, cracks parallel to the corroded bars are produced and the crack width value can be obtained according to expressions of section F.2.2 in this Manual. It is worth pointing out that the mentioned expressions were obtained from the experimental work where the humidity content of the concrete specimens was kept almost constant during the corrosion process. Higher values than those predicted by these formulas can be expected in concrete structures when the concrete close to the bar is submitted to variable humidity conditions.

$$w = 0.05 + ß\,[P_x - P_{x\,o}]\ [w \leq 1.0\text{ mm}] \qquad [f.3]$$

where w is the estimated crack width in mm, P_x is the attack penetration (bar radius decrease) in mm, $P_{x\,o}$ is the attack penetration corresponding to cracking initiation and ß is a coefficient which depends on the position of the bar [ß= 10 for top cast bars and 12.5 for bottom cast bars]. The attack penetration $P_{x\,o}$ needed for the cracking initiation can be estimated by the following expression:

$$P_{x\,o} = (83.8 + 7.4\ c/\phi\ - 22.6\ f_{c,sp}).10^{-3}\ [P_{x\,o} \geq 0] \qquad [f.4]$$

where P_{xo} is the attack penetration in mm, c/ϕ is the cover/diameter ratio and $f_{c,sp}$ is the splitting tensile strength in Mpa, which can be obtained through testing or by the following expressions of Eurocode 2:

$$f_{ct} = 0.3\ f_{ck}^{2/3} \qquad f_{ck} \leq 50\text{ N/mm}^2 \qquad [f.5]$$

$$f_{ct,sp} = f_{ct}/0.9 = 0.333\ f_{ck}^{2/3} \qquad [f.6]$$

Cracking at right angles of the tensile bars is inevitable in both sound and corroded concrete structures subjected to bending, shear, torsion or tension resulting from either direct loading or restrain of imposed deformations, and the crack width could be obtained according to this section. It can be studied using Section 7.3 of Eurocode 2 with some modifications depending on the following parameters:

- Attack penetration $P_{x\,1}$ (tensile bars)
- Attack penetration $P_{x\,2}$ (compression bars)
- f_{bo}, bond strength in sound reinforcement (P_x=0)
- f_b, residual bond strength due to corrosion with attack penetration P_x

The data to be considered as an input in the calculation of the crack width will be the corresponding to the reinforced concrete beam with the following exceptions:

- Reinforcement: the reduced cross sectional area of the rebars at the studied zone of the beam, obtained from the values of attack penetration $P_{x\,1}$ and $P_{x\,2}$, without considering the effect of pitting.

- k : coefficient to take into account the bond properties of reinforcement as the relationship between the residual bond strength and bond strength in sound reinforcement ($k = f_b/f_{bo}$) [f_{bo}, bond strength in sound reinforcement (P_x=0); f_b, residual bond strength due to corrosion with attack penetration P_x]. Coefficient k has to be included into the actual expressions of Eurocode 2 as follows:

$$w_k = s_{rmax}(\varepsilon_{sm} - \varepsilon_{cm}) \qquad [f.28]$$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - 0.4 \dfrac{f_{ct,eff}}{k \ \rho_{p,eff}} \left(1 + \alpha_e \, k \, \rho_{p,eff}\right)}{E_s} \geq 0,6 \frac{\sigma_s}{E_s} \qquad [f.29]$$

$$k \geq 0.5$$

$$s_{rmax} = \frac{\phi_s}{3,6 \, k \, \rho_{p,eff}} \leq \frac{\sigma_s \, \phi_s}{3,6 \, f_{ct,eff}} \qquad [f.30]$$

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